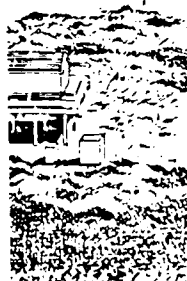
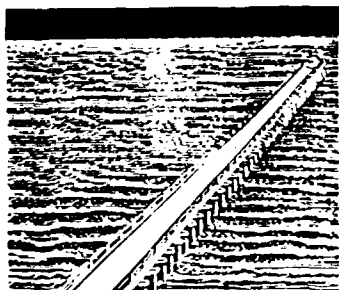




US Army Corps
of Engineers



AD-A214 463



TECHNICAL REPORT CERC-89-12

2

STABILITY OF ARTICULATED CONCRETE MATTRESSES FOR HERBERT HOOVER DIKE IMPROVEMENTS LAKE OKEECHOBEE, FLORIDA

Coastal Model Investigation

by

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Coastal Engineering Research Center

DEPARTMENT OF THE ARMY

Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199



September 1989

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19. ABSTRACT (Continue on reverse if necessary and identify by block number) The purpose of the model investigation reported herein was to investigate the suitability of articulated concrete mattresses (ACM's) for use at Lake Okeechobee, FL. Due to the absence of significant experience in applying this method of slope protection in a wave environment and the lack of appropriate design criteria, it was decided to investigate the stability response of the ACM's via a large-scale, two-dimensional coastal model. Based on tests conducted, it was concluded that for one layer of ACM's: a. The maximum stable wave height is 3 ft if the mat is secured by top anchors only. b. An addition of three rows of anchors near the still water level (swl) and two rows of toe anchors provides stability for up to 4-ft waves.					
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19. ABSTRACT (Continued).

- c. Anchors would be required between every other block and every other row from the water surface to the toe in order to achieve stability for the 5- and 6-ft waves.
- d. Five- and eight-hundred-pound toe stones proved to be stable for 3- and 4-ft waves, respectively.
- e. Additional toe stone reduces the maximum runup by about 0.5 ft.

For two layers of ACM's it was concluded that:

- a. The maximum stable wave height is 4 ft if the mats are secured by top anchors only.
- b. With the toe stabilized by anchors or stone, localized lifting of the mats in the vicinity of the swl can be expected for 5- and 6-ft waves.
- c. Construction of a 3-ft-deep toe trench will significantly improve stability of the toe stone; however, stones loosened by the higher wave heights have the potential to roll upslope and downslope and may damage the mats.
- d. An additional single layer of mats on the overbank will reduce maximum runup by about 1 ft.
- e. Bedding stone movement can be expected for wave heights in excess of 4 ft.
- f. Erosion of the sand embankment will be initiated by 2-ft waves if a bedding or filtering medium is not provided.

For three layers of ACM's, it was concluded that ACM's are completely stable for 2-, 3-, and 4-ft waves. However, minor lifting of the toe can be expected for the 5- and 6-ft waves if toe anchors are not used.

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PREFACE

The US Army Engineer District, Jacksonville (SAJ), requested the US Army Engineer Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC) to conduct a study on the use of articulated concrete mattresses on the Herbert Hoover Dike in Lake Okeechobee, FL. Funding authorization by SAJ was granted by Intra-Army Order No. BEA00-304L2-OGJ14, dated 21 November 1988.

This study was conducted under the general direction of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Chief and Assistant Chief, CERC, respectively; and under the direct guidance of Messrs. C. Eugene Chatham, Chief, Wave Dynamics Division, and D. Donald Davidson, Chief, Wave Research Branch. Tests were conducted in the Wave Dynamics Division under the direction of Mr. R. D. Carver, Principal Investigator, and by Mrs. B. J. Wright and Mr. C. Lewis, Engineering Technicians. This report was prepared by Messrs. Carver and Davidson and Mrs. Wright.

Commander and Director of WES during the publication of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI
(metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre
square feet	0.09290304	square metres

STABILITY OF ARTICULATED CONCRETE MATTRESSES FOR HERBERT HOOVER
DIKE IMPROVEMENTS, LAKE OKEECHOBEE, FLORIDA

Coastal Model Investigation

PART I: INTRODUCTION

Background

1. At the request of the US Army Engineer District, Jacksonville (SAJ), an investigation of the use of articulated concrete mattresses (ACM's) for levee protection at Lake Okeechobee, FL, was conducted by the Waterways Experiment Station's Coastal Engineering Research Center. Lake Okeechobee is located in south-central Florida (Figure 1). Prior to 1928, local interests provided low levees around the southern shore of Lake Okeechobee. These levees were raised and extended during the 1930's, under the authority of the River and Harbor Act of July 3, 1930. In the 1960's, the levees were again raised and extended under the authority of the Flood Control Act of June 30, 1948. Under provisions of this authority, riprap protection was provided as a salient feature of the levee system with protection constructed when and where needed as indicated by operating expenses.

2. In the early years of the project, the lake was operated between elevations +12.5 and +15.5 ft National Geodetic Vertical Datum (NGVD).^{*} Over a period of time, the operational pool elevation was increased. In 1978 it was established at el +15.5 to +17.5. Prior to raising the pool elevation to this level, about 50 miles** of levees were protected with riprap or other type structural works. However, with the operational lake levels adopted in 1978, the water surface has risen above most of the existing revetments and wave action on the exposed levee has caused significant erosion.

Purpose of Model Investigation

3. A Value Engineering study suggests that required levee protection can be provided by ACM's, similar to that used by the Memphis District on the

* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

** A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

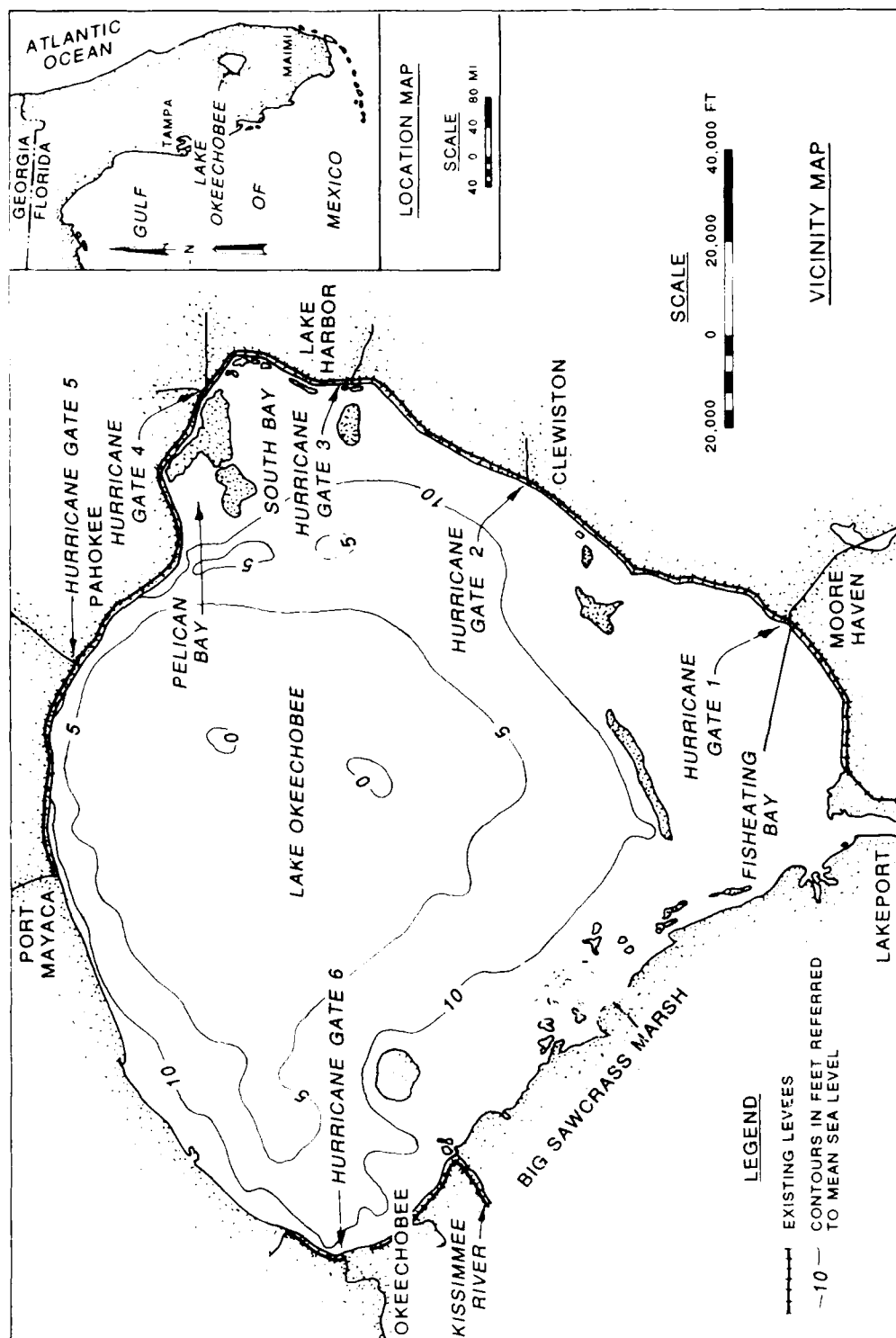


Figure 1. Location and vicinity map

Mississippi River, at a significant cost reduction over existing revetments. However, there is little experience in applying this method of slope protection in a wave environment. Due to the absence of significant experience and an appropriate design criteria, it was decided that a physical model investigation offered the best means of investigating the ACM's suitability for application at Lake Okeechobee. Specifically, the purpose of the model investigation was to obtain qualitative indications of mat movement, toe-stone stability, and wave runup for various incident wave conditions.

PART II: THE MODEL

Model-Prototype Scale Relationships

4. Tests were conducted at a geometrically undistorted scale of 1:10, model to prototype. Based on Froude's model law (Stevens et al. 1942)* and the linear scale of 1:10, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

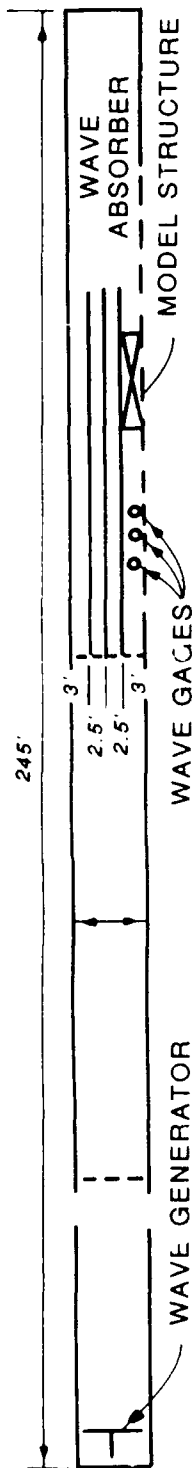
<u>Characteristic</u>	<u>Dimension</u>	<u>Model-Prototype Scale Relation</u>
Length	L	$L_r = 1:10$
Area	L^2	$A_r = L_r^2 = 1:100$
Volume	L^3	$V_r = L_r^3 = 1:1000$
Time	T	$T_r = L_r^{1/2} = 1:3.33$

Both geometric and dynamic similitude were ensured by making the model ACM's volumetrically reduced while maintaining a one-to-one material density relationship, i.e., the prototype concrete unit weight of 135 pcf was reproduced with 135 pcf concrete in the model. Fresh water ($\gamma = 62.4$ pcf) was used in the model, thus giving the desired model-prototype specific gravity ratio of unity.

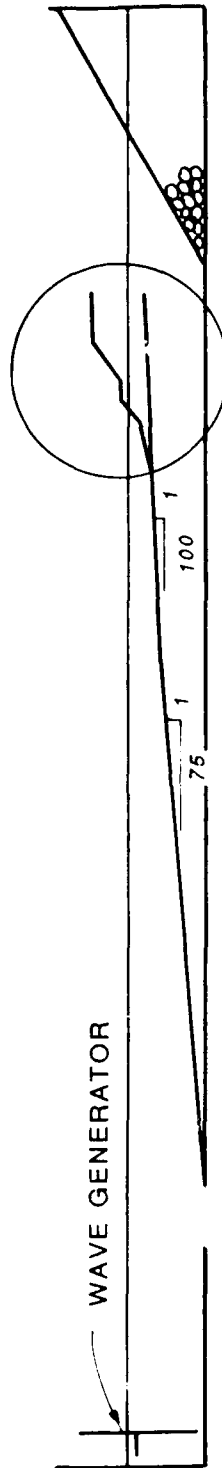
Test Equipment and Facilities

5. All tests were conducted in a concrete wave flume that measured 11-ft wide and 245-ft long. The cross section of the tank in the vicinity of the structure was partitioned into two 3-ft-wide channels and two 2.5-ft-wide channels (Figure 2). Test sections were constructed in the glass-sided, 3-ft-wide portion of the flume. The remaining empty channels acted as absorbers. Irregular waves were generated by a hydraulically actuated piston-driven wave board. Test sections were installed approximately 190 ft from the wave board.

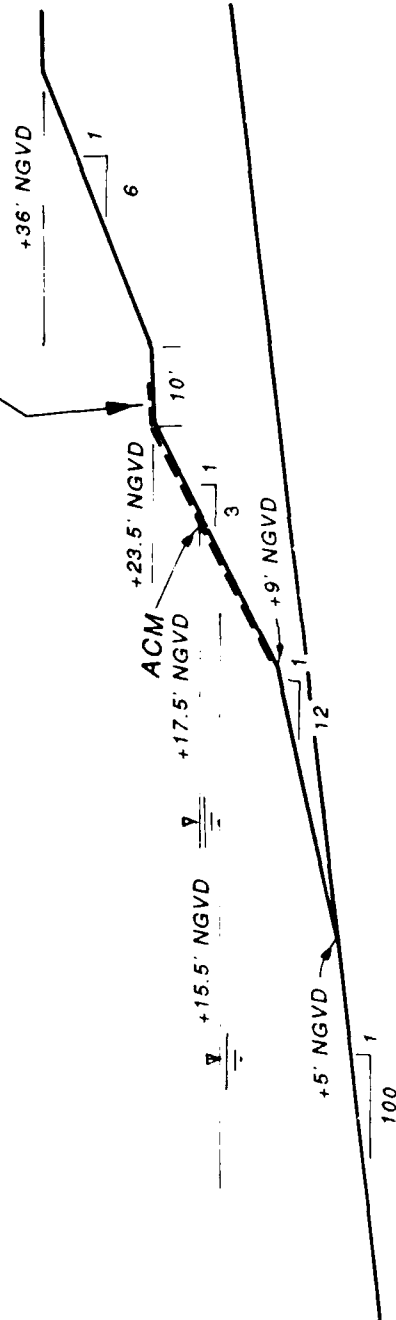
* J. C. Stevens, C. E. Bardsley, E. W. Lane, L. G. Straub, C. A. Wright, J. B. Tiffany, and J. E. Warnock. 1942. "Hydraulic Models," Manuals of Engineering Practice, American Society of Civil Engineers, Vol. 25, pp 59-68.



PLAN VIEW



FLUME SIDE VIEW



ENLARGED VIEW

Figure 2. Flume layout and test section

6. Wave data were collected on electrical resistance wave gages. Wave signal generation and data acquisition were controlled using a DEC MicroVax I computer. Wave data analyses were accomplished using a DEC VAX 11/750 computer.

Model Approach and Setup

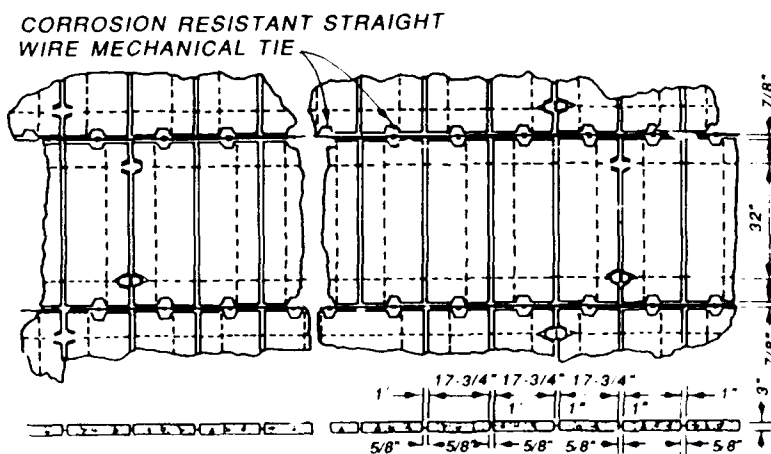
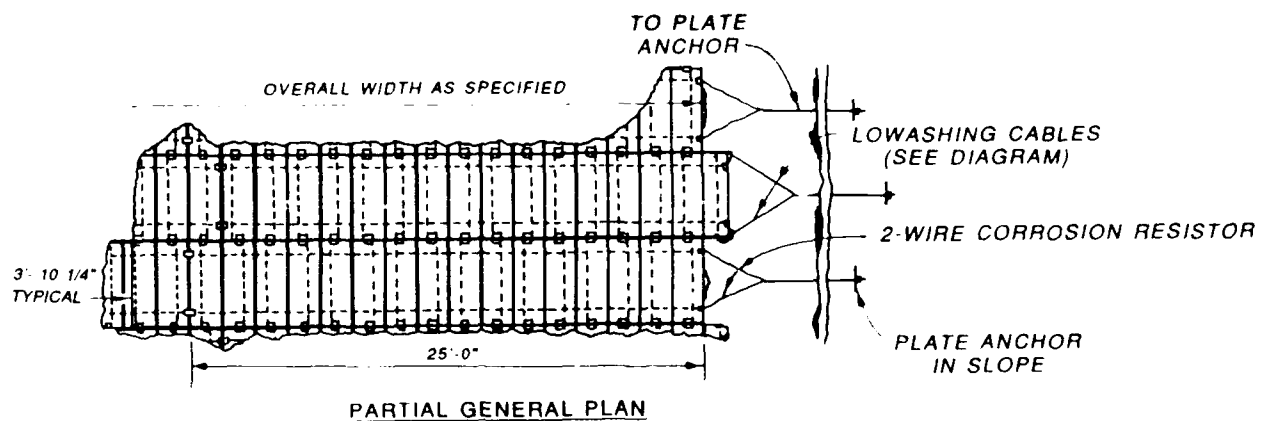
7. The approach of the study was to investigate the stability response of the ACM's via a large-scale, two-dimensional (2-D) coastal model. Model ACM's were observed under simulated wave attack to determine qualitative mat movement and other factors that could be used to help determine their suitability for use in the anticipated prototype wave environment.

8. It was determined that the prototype mat(s) should measure about 50 ft upslope and downslope and be continuously connected along the slope. Since the model flume width (3 ft) was equivalent to 30 ft prototype, model mats were constructed which correctly reproduced both the geometry and weight of 27-ft-wide by 50-ft-long prototype mats. Details of a typical prototype mat are shown in Figure 3.

9. All mats were secured in the model by top anchors at the berm (el = +23.5) and extended downslope to a toe elevation of +9.0. Initial test plans did not incorporate toe protection (Figure 4a), whereas subsequent test plans used onslope toe protection (Figure 4b) or simulated placement of the toe stone in a 3-ft-deep trench (Figure 4c). Detailed descriptions of individual plans are presented in Part III.

10. Initial tests were conducted to determine under what wave conditions the single, double, and triple mat systems would move and what stability improvements could be achieved with toe stone and/or ground anchors. Tests also were conducted with a stone bedding layer installed under the two-layer mat system to determine if leaching of the bedding stone through the mats or downslope migration under the mats would be a potential problem. The size distribution of the prototype bedding material was as follows:

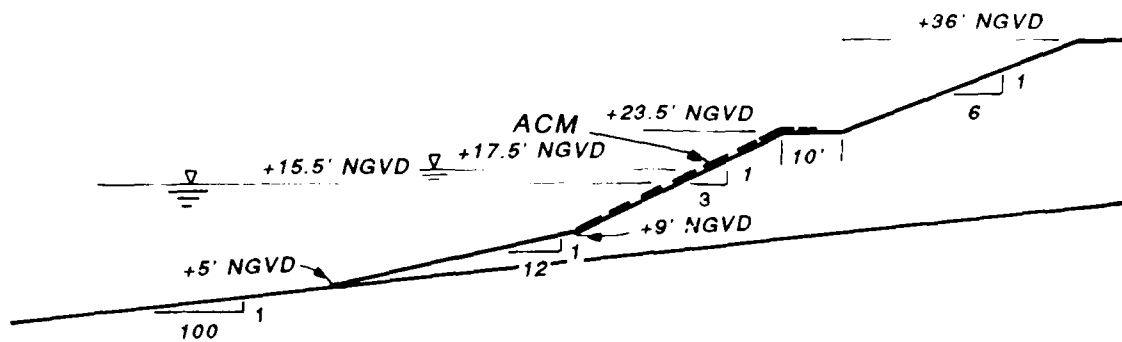
<u>Sieve Size</u>	<u>Percent Finer</u>
4.00 in.	100
3.00 in.	90
1.50 in.	75
0.75 in.	55
No. 4	42
No. 16	22
No. 50	5
No. 200	0



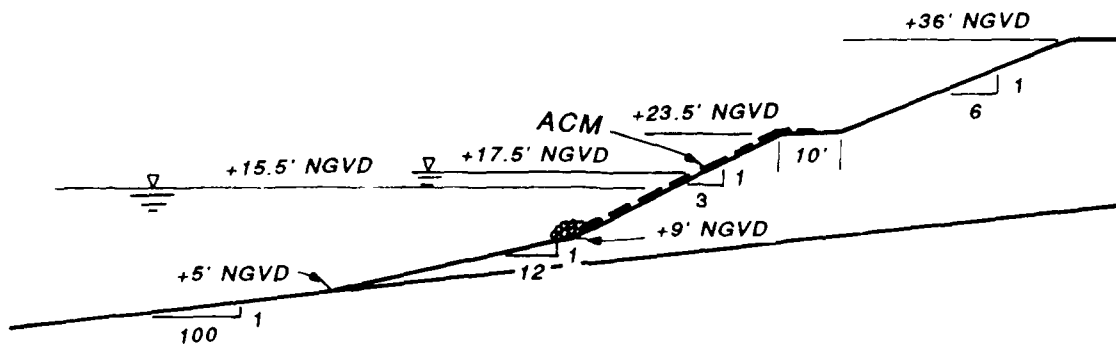
DETAIL SECTION OF MATTRESS

12" 0' 1' 2' 3' 4' 5'

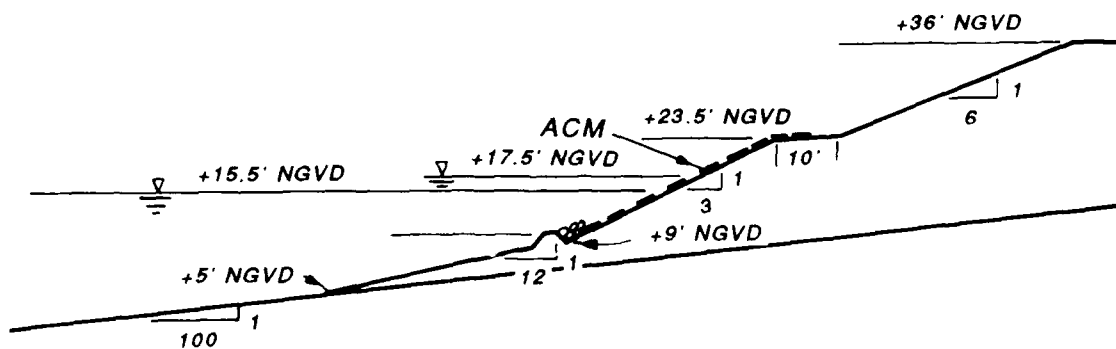
Figure 3. Typical ACM



a. No toe protection



b. Onslope toe stone



c. Toe trench filled with stone

Figure 4. ACM toe conditions tested

PART III: TESTS AND RESULTS

Selection of Test Conditions

11. Based on information provided by the SAJ that all prototype installations would be in shallow water, tests were conducted with a Texel, Marsen, and Arsloe (TMA) spectrum using a peak period (T_p) of 5.8 sec. Initial plans were to be tested at a still-water level (swl) of +17.5 with the final plan(s) to be checked for a lower swl (+15.5) as needed. The wave basin was calibrated for wave height (H_{mo}) values of 2, 3, 4, 5, and 6 ft measured about 100 ft (prototype) in front of the test sections. Testing with incremental wave heights allowed the threshold of mat and toe-stone movement to be defined.

Plans Tested and General Results

12. A total of 14 plans were tested. All mats were secured by top anchors at the berm (el = +23.5) and extended downslope to a toe elevation of +9. All plans were tested at an swl of +17.5 and Plan 2BTA was also checked at an swl of +15.5. Details of the plans tested and general results are as follows.

Plan 1 (Photos 1 and 2)

13. Plan 1 consisted of a one-layer mat secured only by top anchors. This plan was stable for 2- and 3-ft waves; however, the 4-ft waves produced some lifting near the toe, and both the 5- and 6-ft waves caused the mat to roll upslope. Violent motions associated with the 5- and 6-ft waves caused some blocks near the toe to crack and/or break loose from the mat. Maximum observed runup was el +29. Runup values for all wave conditions investigated are presented in Table 1.

Plan 1A

14. Plan 1A was the same as Plan 1 except three rows of anchors were added near the swl. Test results were similar to Plan 1.

Plan 1B (Photos 3 and 4)

15. Plan 1B was the same as Plan 1A except two rows of toe anchors were added. Stability improved slightly over previous plans; however, excessive lift still was observed in the unsecured area between the swl

and toe for 5- and 6-ft waves. Maximum observed runup was the same as the previous plans (+29).

Plan 1C (Photos 5 and 6)

16. Plan 1C was similar to Plan 1B except additional anchors were added at alternate joints from the swl to the toe. As expected, this plan proved stable for all wave heights. Maximum runup was unchanged.

Plan 1D (Photos 7 and 8)

17. Plan 1D was secured by top anchors and a 6-ft-wide, two-layer berm of 500-lb toe stone. This plan proved to be stable for 2- and 3-ft waves; however, the 4-ft waves produced significant movement of the toe stone and the 5- and 6-ft waves caused excessive numbers of toe stone to roll upslope and downslope. Excessive lifting of the mat also was observed in the vicinity of the swl for 5- and 6-ft waves. Maximum runup was reduced to +28.5.

Plan 1E (Photos 9 and 10)

18. Plan 1E was the same as Plan 1D except the toe stone weight was increased to 800 lb. Tests proved the plan to be stable for up to 4-ft waves; however, the 5- and 6-ft waves still caused excessive movement of toe stone upslope and downslope and produced significant lifting of the mat in the vicinity of the swl. Maximum runup was unchanged at +28.5.

Plan 2 (Photos 11 and 12)

19. Plan 2 consisted of two layers of mats laced together at intervals of every other block and every other row and secured by top anchors. This plan was stable for 2-, 3-, and 4-ft waves; however, both the 5- and 6-ft waves produced lifting of the toe. Maximum observed runup was the same as Plans 1 through 1C (+29).

Plan 2A (Photos 13 and 14)

20. Plan 2A was the same as Plan 2 except two rows of toe anchors were added. Stability improved; however, slight localized lifting of the mat was observed in the vicinity of the swl for the 5- and 6-ft waves. Maximum runup was unchanged at +29.

Plan 2B (Photos 15 and 16)

21. Plan 2B was secured by top anchors and a 6-ft-wide, two-layer berm of 800-lb toe stone. This plan proved to be stable for 2-, 3-, and 4-ft waves; however, the 5- and 6-ft waves caused excessive movement of toe stone upslope and downslope. Very minor localized lifting of the mats in the

vicinity of the swl was observed during attack of the 5- and 6-ft waves. Maximum runup was reduced slightly to +28.5.

Plan 2BF (Photos 17 and 18)

22. Plan 2BF was the same as Plan 2B except a 1-ft-thick layer (prototype) of bedding stone was placed under the mats. As would be expected, the dynamic response of the mats and toe stone was the same as Plan 2B. The bedding stone proved to be stable for the 2-, 3-, and 4-ft waves; however, the 5-ft waves initiated downslope migration under the mats, and the 6-ft waves produced failure of the bedding stone.

Plan 2BT (Photos 19-22)

23. Plan 2BT was similar to Plan 2B except 500-lb toe stone was placed in a 3-ft-deep trench in an effort to improve stability. Also, a single mat was placed on the overbank above the existing two mats in an effort to reduce runup. Stability was achieved for 2-, 3-, and 4-ft waves; however, the 5- and 6-ft waves caused significant movement of toe stone upslope and downslope. The toe trench significantly improved stability in that the 500-lb stone exhibited a stability response similar to that observed for the 800-lb stone (tested in Plan 2B without a toe trench). Some localized lifting of the mats in the vicinity of the swl was observed during attack of the 5- and 6-ft waves. Maximum runup was reduced to +27.5.

Plan 2BTA (Photos 23-26)

24. Plan 2BTA was the same as Plan 2BT except the toe stone weight was increased to 800 lb and three rows of anchors were added in the vicinity of the swl. As anticipated, toe-stone stability improved for the 5-ft waves; however, the 6-ft waves still caused significant movement of the toe stone upslope and downslope. Also, very minor localized lifting of the mats was observed between the areas secured by anchors and toe stone for the 5- and 6-ft waves. A check test at the +15.5 swl showed slightly greater stability than was observed at the +17.5 swl. Maximum observed runup was the same as Plan 2BT (+27.5).

Plan 2BTS (Photos 27-30)

25. Plan 2BTS was the same as Plan 2BT except the toe-stone weight was 800 lb and the mats were placed over a 2.5-ft-thick layer of sand in an effort to obtain a qualitative indication of sand movement. Attack of the 2-ft waves caused about half of the sand layer to erode between the swl and +21. The 3-ft waves removed the sand layer between the swl and +22. Attack of 4-ft

waves produced complete removal of sand above the swl as evidenced in Photos 27 and 28.

Plan 3 (Photos 31 and 32)

26. Plan 3 consisted of three layers of mats laced together and secured by top anchors. This plan proved stable for 2-, 3-, and 4-ft waves; however, both the 5- and 6-ft waves caused minor lifting of the toe. Maximum runup was the same as Plan 2 (+29).

Discussion

27. It was beyond the scope of the present investigation to model the compressive strength (minimum = 2,000 psi) of the prototype concrete. The model concrete had a compressive strength significantly greater than if it had been structurally simulated; therefore, breakage of the model mats observed during testing of Plans 1 and 1A with 5- and 6-ft waves probably indicates a major prototype problem should this anchoring arrangement be exposed to wave conditions of this magnitude. Also, mats may be damaged by impacts from toe stones for those wave conditions severe enough to produce toe-stone movement.

28. During the course of testing, interest in the survivability (ability to withstand wave conditions in excess of the selected design conditions) of the two-layer plans arose. Plan 2BTA, one of the better alternatives tested, was exposed to 7-ft waves at an swl of +19.5 and movement of both the mats and toe stone was similar to that observed for the 6-ft waves.

29. It should be noted that the after-testing photographs presented in this report cannot always depict the level of damage or potential damage observed during the course of testing. A more detailed understanding of the dynamic response of the various plans tested herein can be gained by viewing video tapes of the tests.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

30. Based on tests and results reported herein, it was concluded that for one layer of ACM's:

- a. The maximum stable wave height is 3 ft if the mat is secured by top anchors only.
- b. An addition of three rows of anchors near the swl and two rows of toe anchors provides stability for up to 4-ft waves.
- c. Anchors would be required between every other block and every other row from the water surface to the toe to achieve stability for the 5- and 6-ft waves.
- d. Five- and eight-hundred-pound toe stone proved to be stable for 3- and 4-ft waves, respectively.
- e. Addition of toe stone reduces the maximum runup by about 0.5 ft.

31. For two layers of ACM's it was concluded that:

- a. The maximum stable wave height is 4 ft if the mats are secured by top anchors only.
- b. With the toe stabilized by anchors or stone, localized lifting of the mats in the vicinity of the swl can be expected for 5- and 6-ft waves.
- c. Construction of a 3-ft-deep toe trench will significantly improve stability of the toe stone; however, stones loosened by the higher wave heights have the potential to roll upslope and downslope and may damage the mats.
- d. An additional single layer of mats on the overbank will reduce maximum runup by about 1 ft.
- e. Bedding stone movement can be expected for wave heights in excess of 4 ft.
- f. Erosion of the sand embankment will be initiated by 2-ft waves if a bedding or filtering medium is not provided.

32. Three layers of ACM's are recommended as the most stable alternative of the alternatives investigated. The three layers of ACM's are completely stable for 2-, 3-, and 4-ft waves. Any toe lifting observed for waves of 5 and 6 ft were eliminated by the use of either ground anchors or toe stone.

Table 1
Maximum Observed Runup Values

<u>Plan No.</u>	<u>Maximum Runup for Indicated Wave Height, ft</u>				
	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>6</u>
1	24.0	25.5	27.0	28.0	29.0
1A	*	*	27.0	28.0	29.0
1B	*	*	27.0	28.0	29.0
1C	*	*	27.0	28.0	29.0
1D	24.0	25.5	27.0	28.0	28.5
1E	*	25.0	26.5	27.5	28.5
2	22.5	25.0	27.0	28.0	29.0
2A	*	*	27.0	28.0	29.0
2B	*	*	27.0	27.5	28.5
2BT	*	24.5	25.5	26.5	27.5
2BTA	*	24.5	25.5	26.5	27.5
2BTS	21.0	22.0	23.0	*	*
3	*	*	27.0	28.0	29.0

* This condition was not tested.

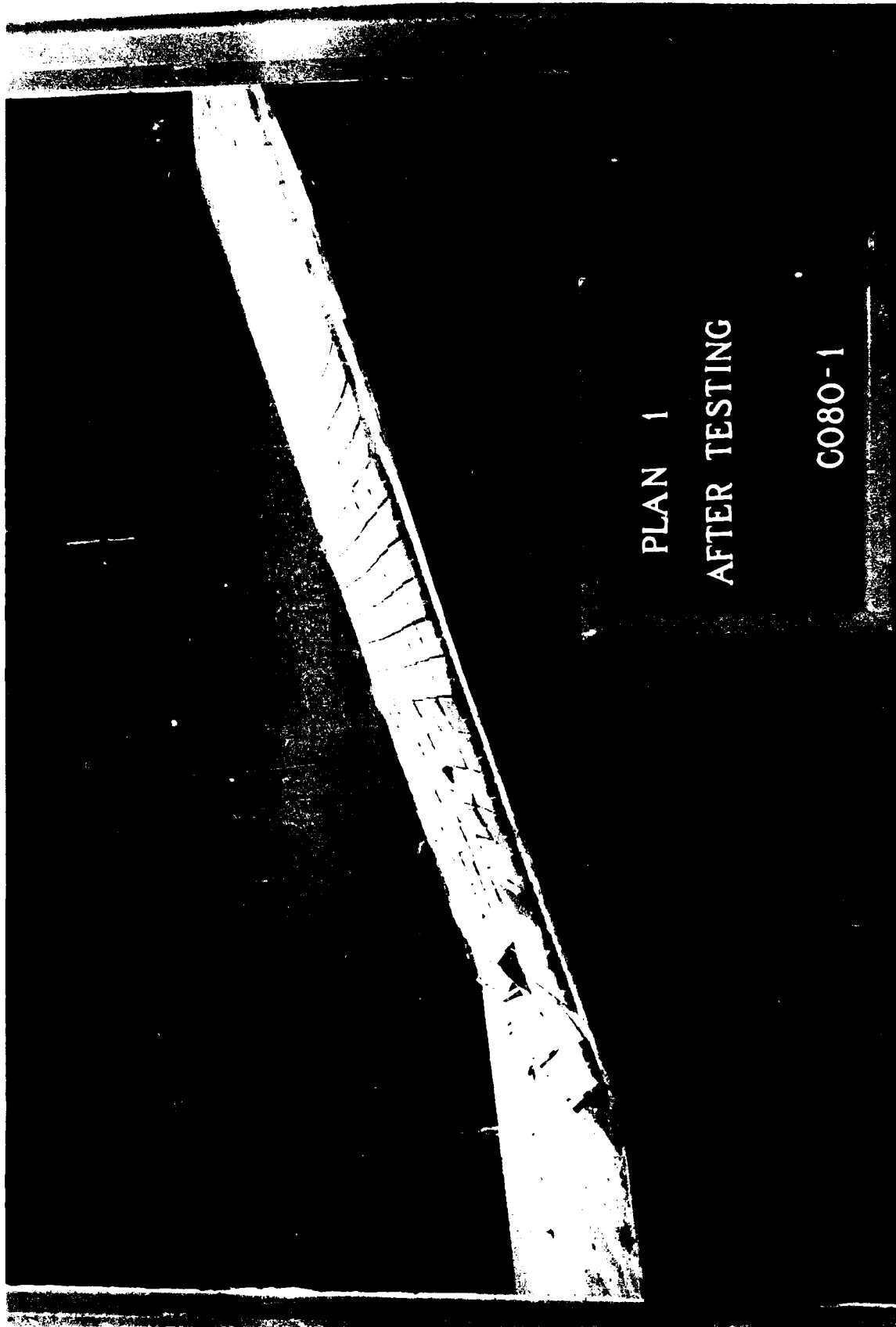


Photo 1. Side view of Plan 1 after wave attack



Photo 2. Sea-side view of Plan 1 after wave attack

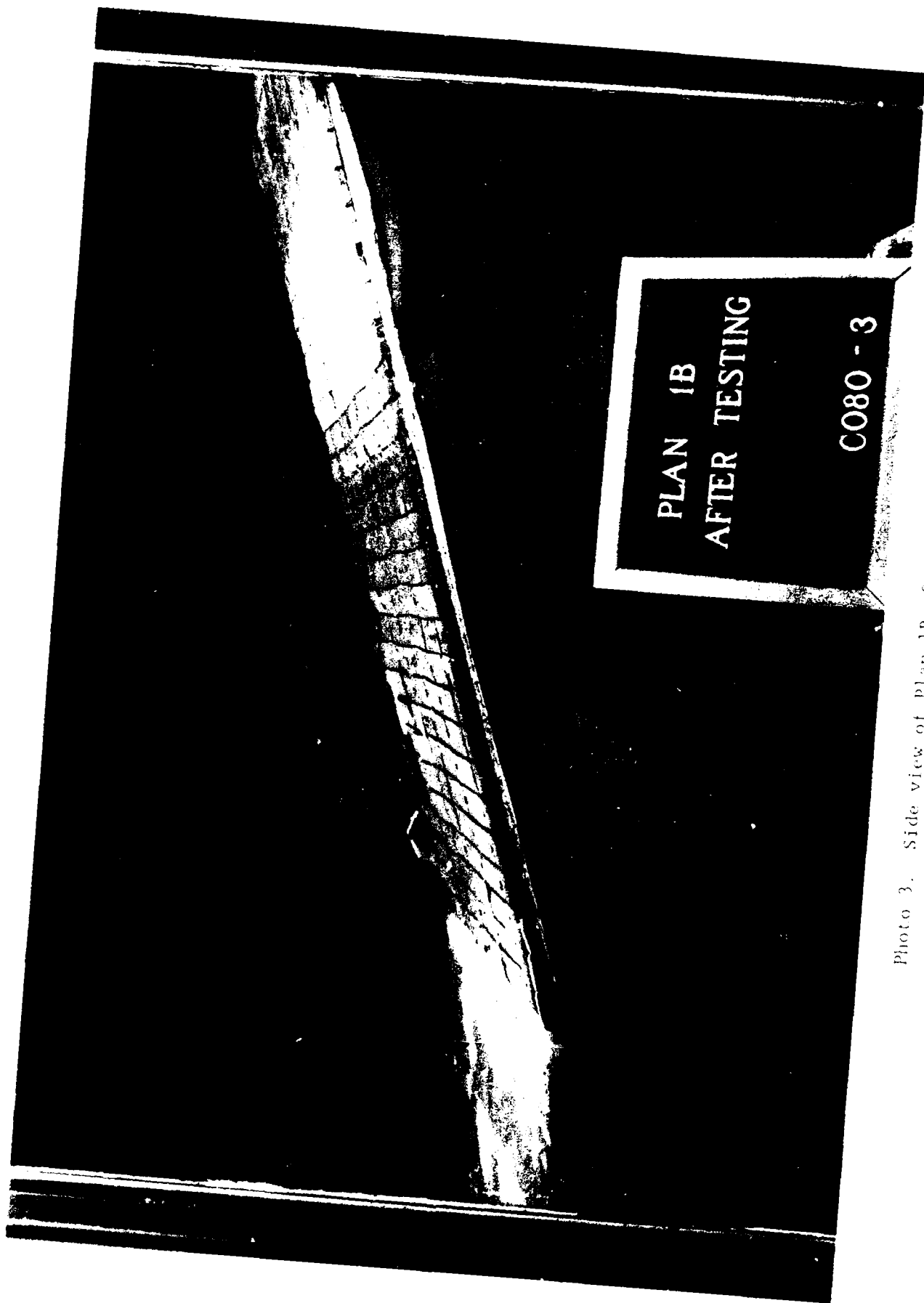


Photo 3. Side view of Plan 1B after wave attack



Photo 1. Sea-side view of Plan 1B after wave attack

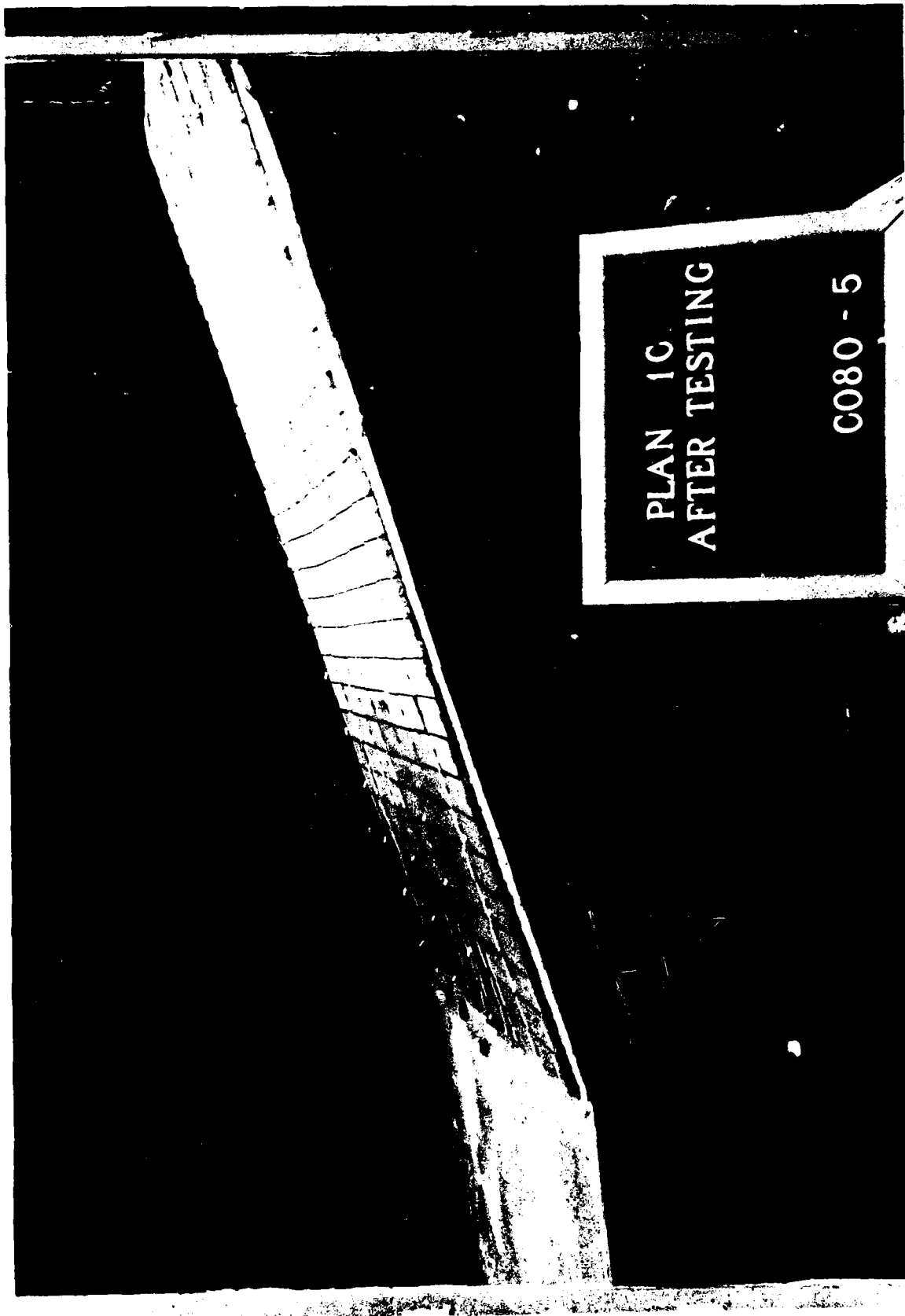


Photo 5. Side view of Plan 1C after wave attack



Photo 6. Sea-side view of Plan IC after wave attack

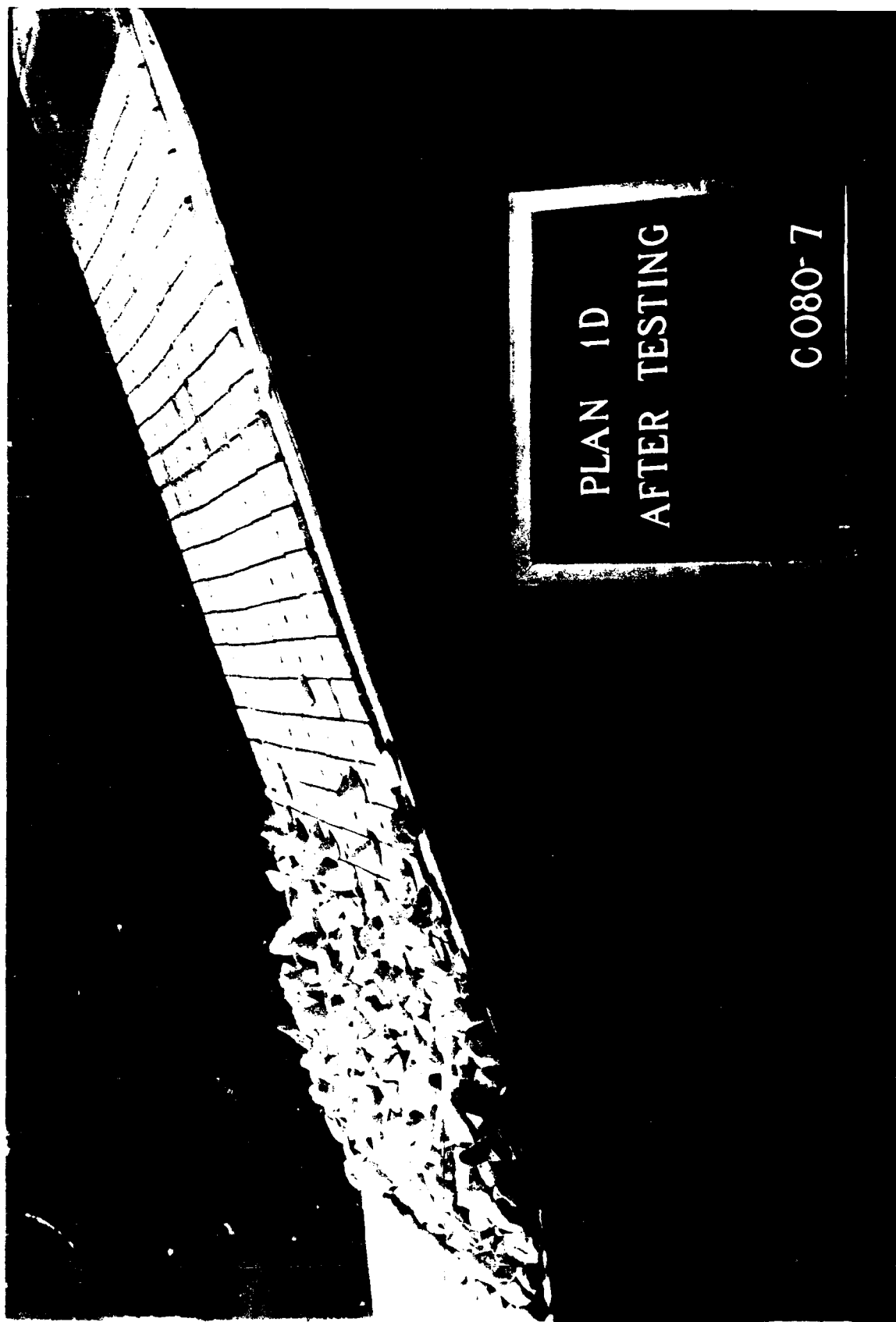


Photo 7. Side view of Plan 1D after wave attack

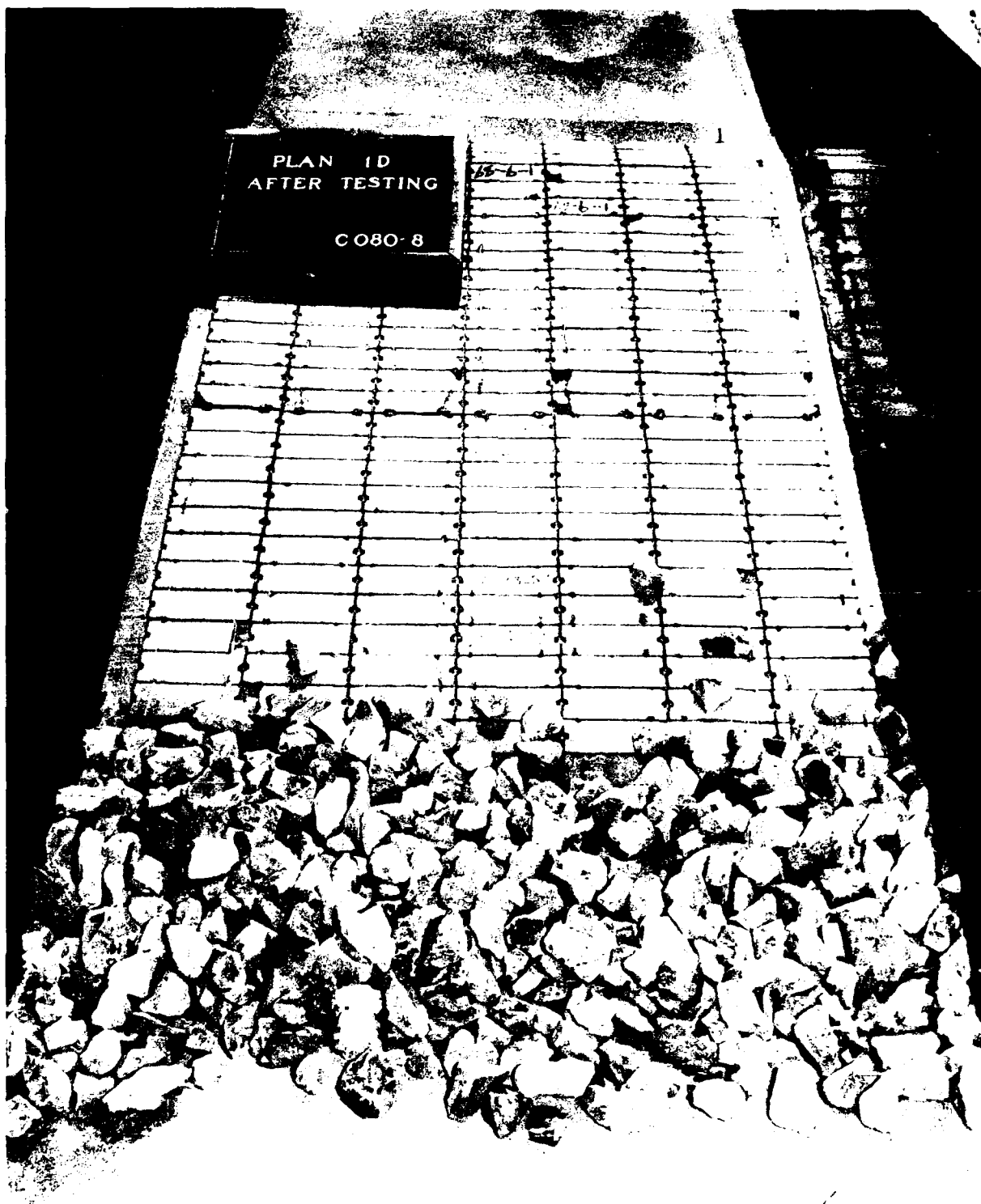


Photo 8. Sea-side view of Plan 1D after wave attack

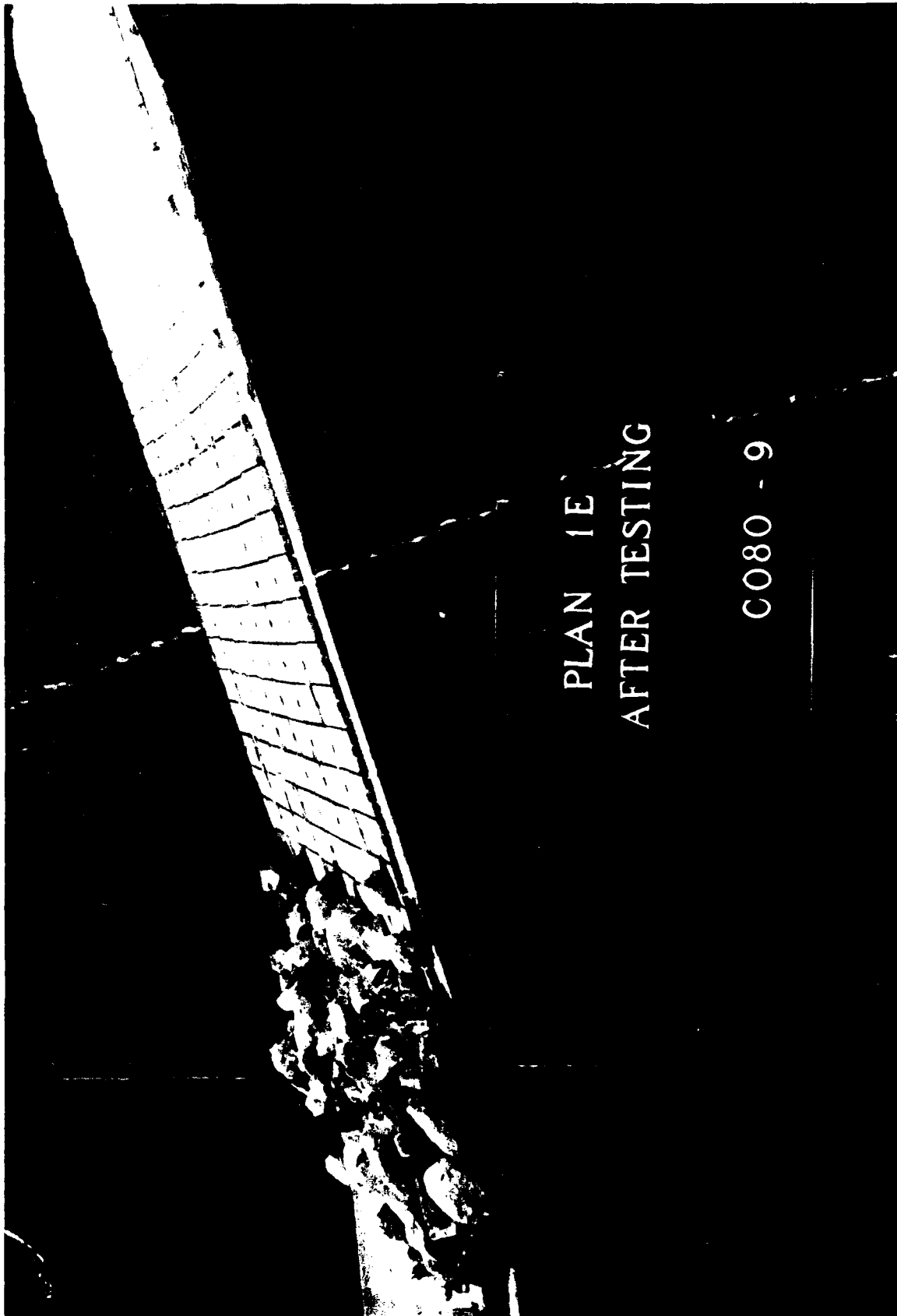


Photo 9. Side view of Plan 1E after wave attack



Photo 10. Sea-side view of Plan 1E after wave attack

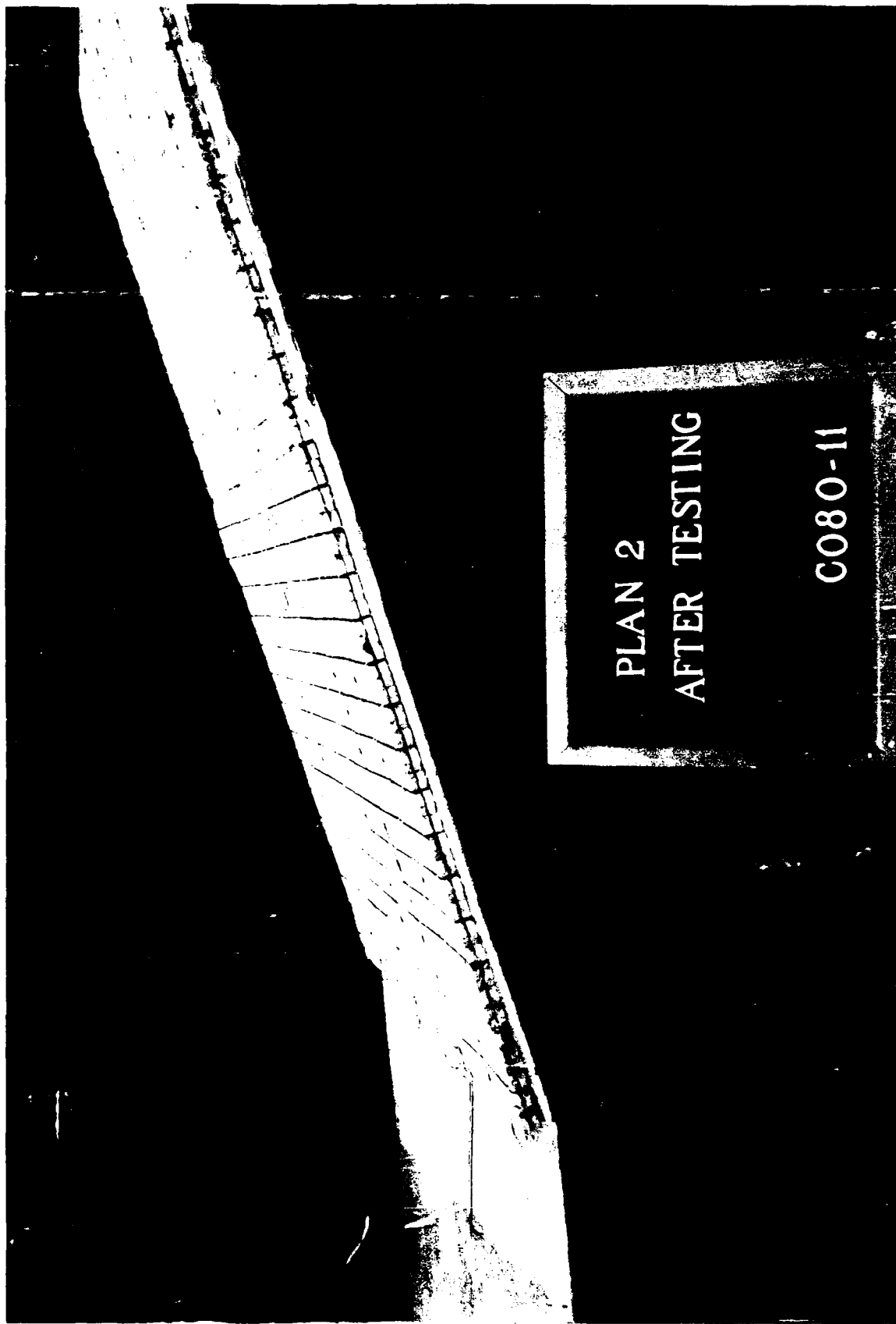


Photo 11. Side view of Plan 2 after wave attack

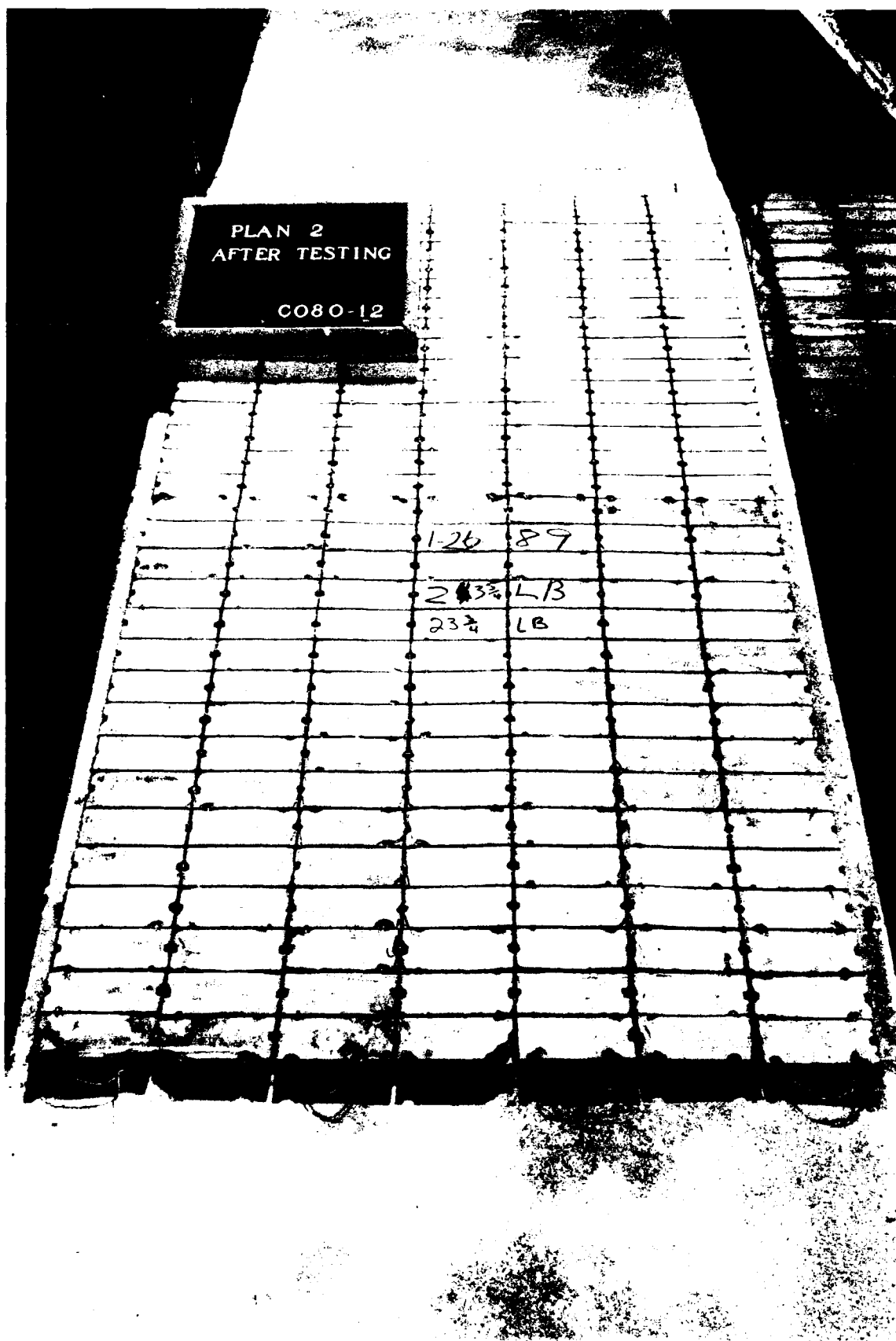


Photo 12. Sea-side view of Plan 2 after wave attack

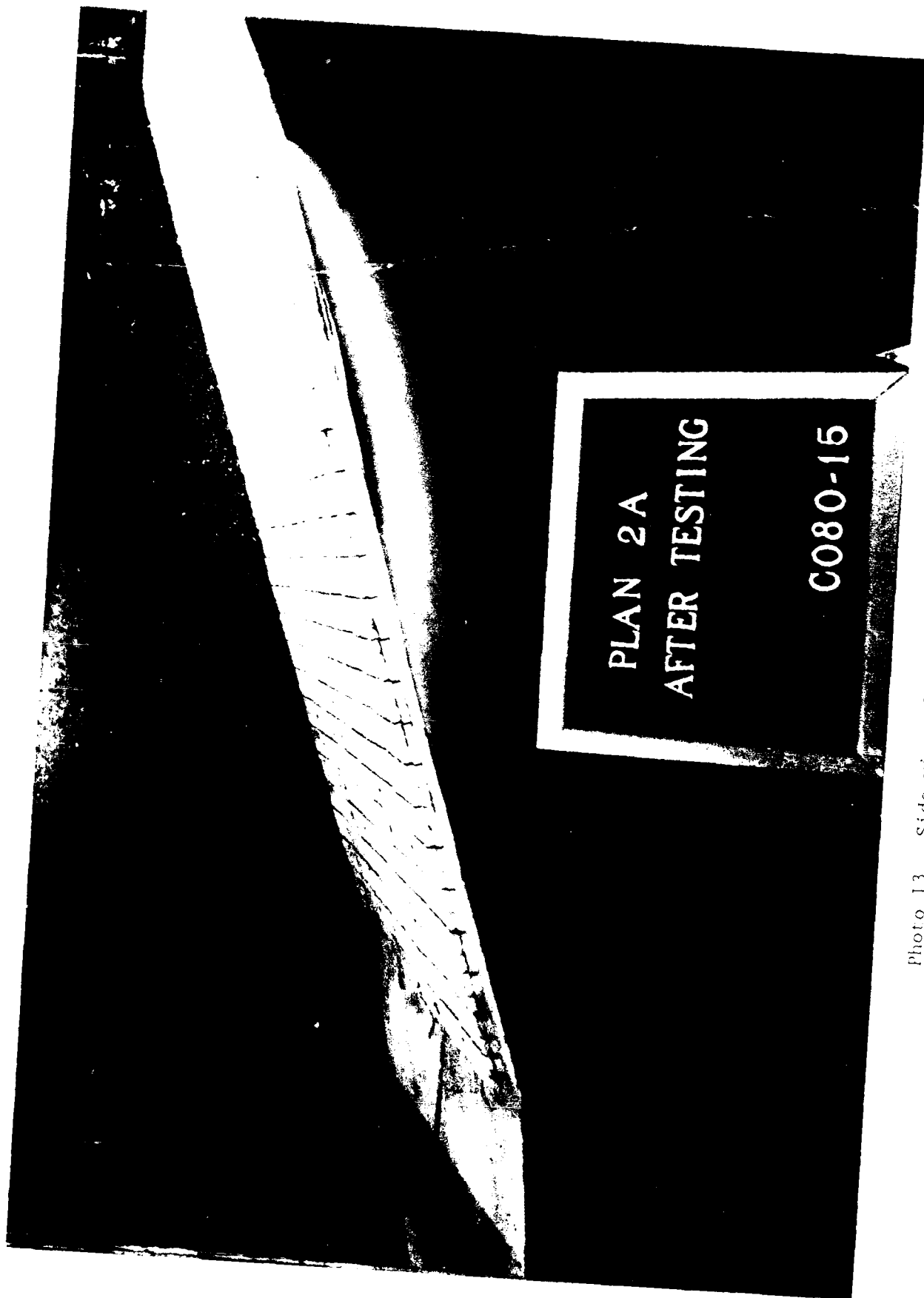


Photo 13. Side view of Plan 2A after wave attack

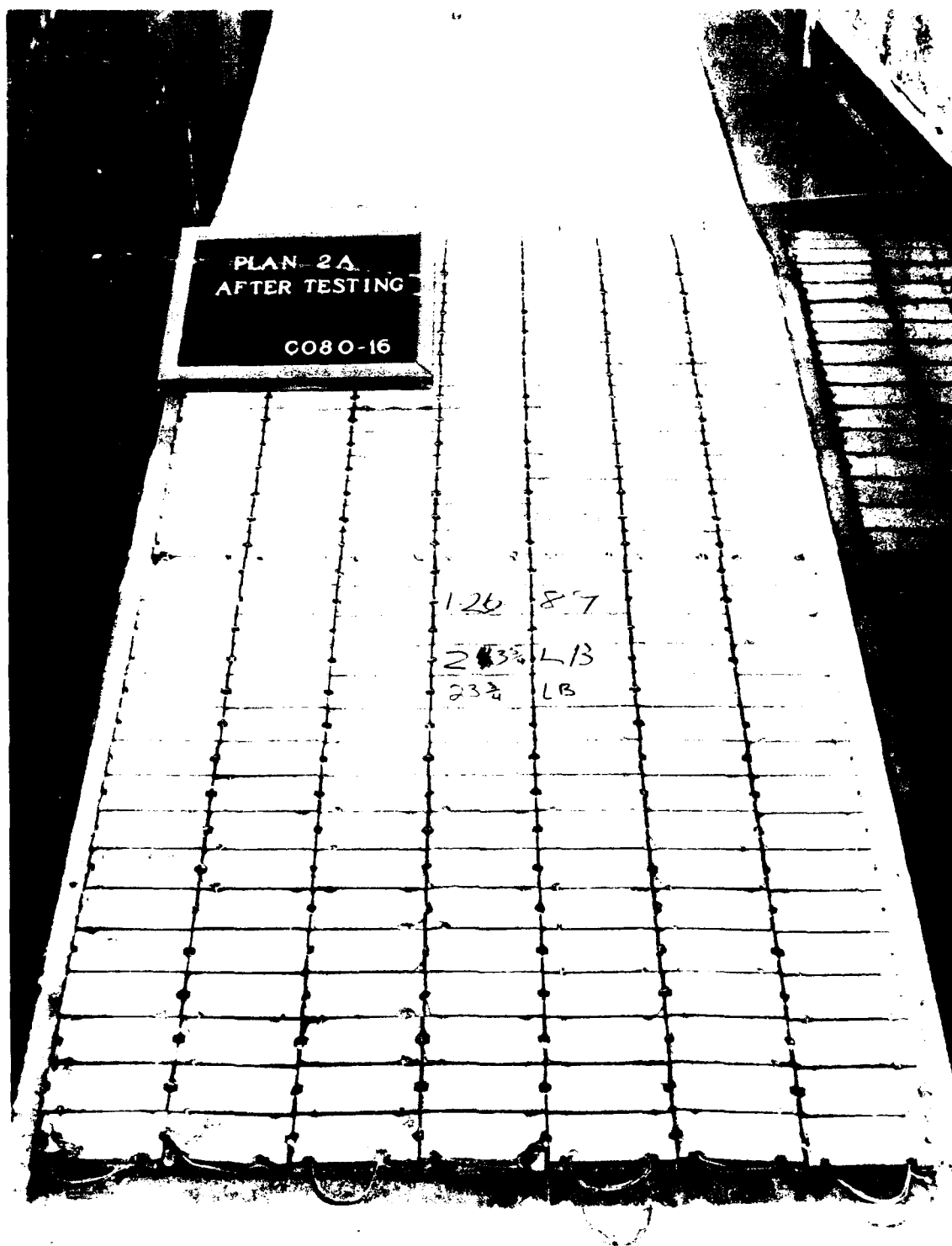
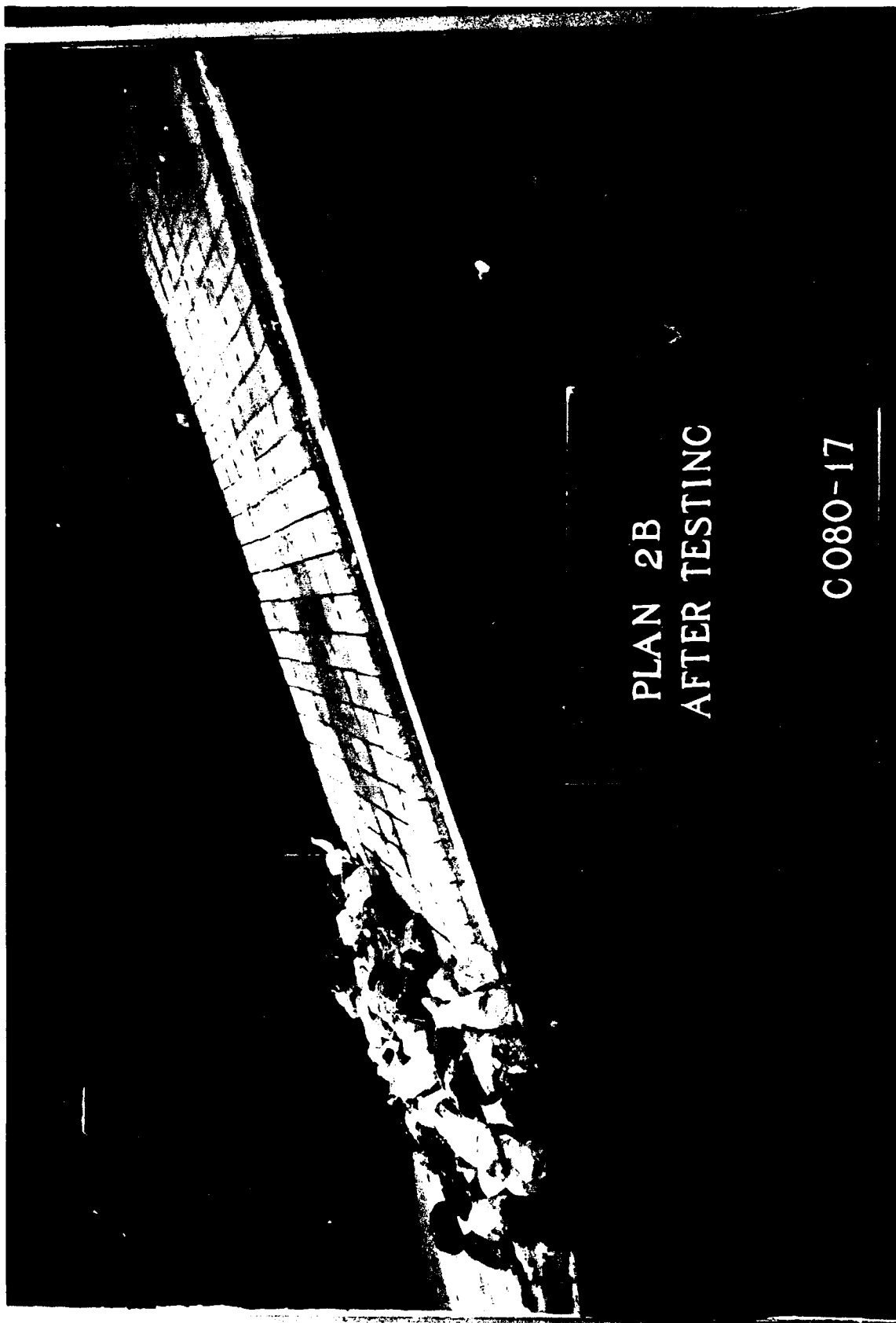


Photo 14. Sea-side view of Plan 2A after wave attack



PLAN 2B
AFTER TESTING

C080-17

Photo 15. Side view of Plan 2B after wave attack



Photo 16. Sea-side view of Plan 2B after wave attack

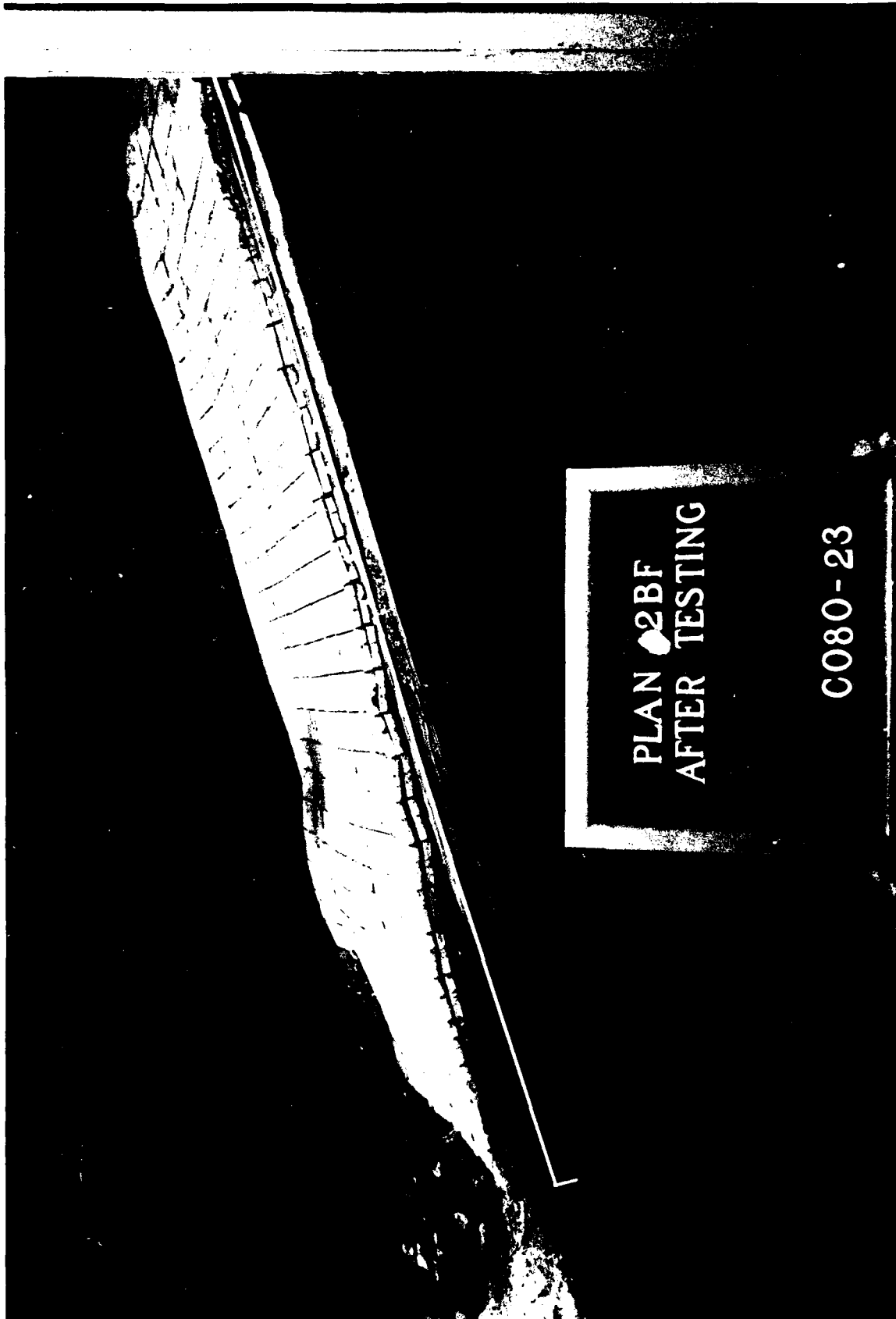


Photo 17. Side view of Plan 2BF after wave attack

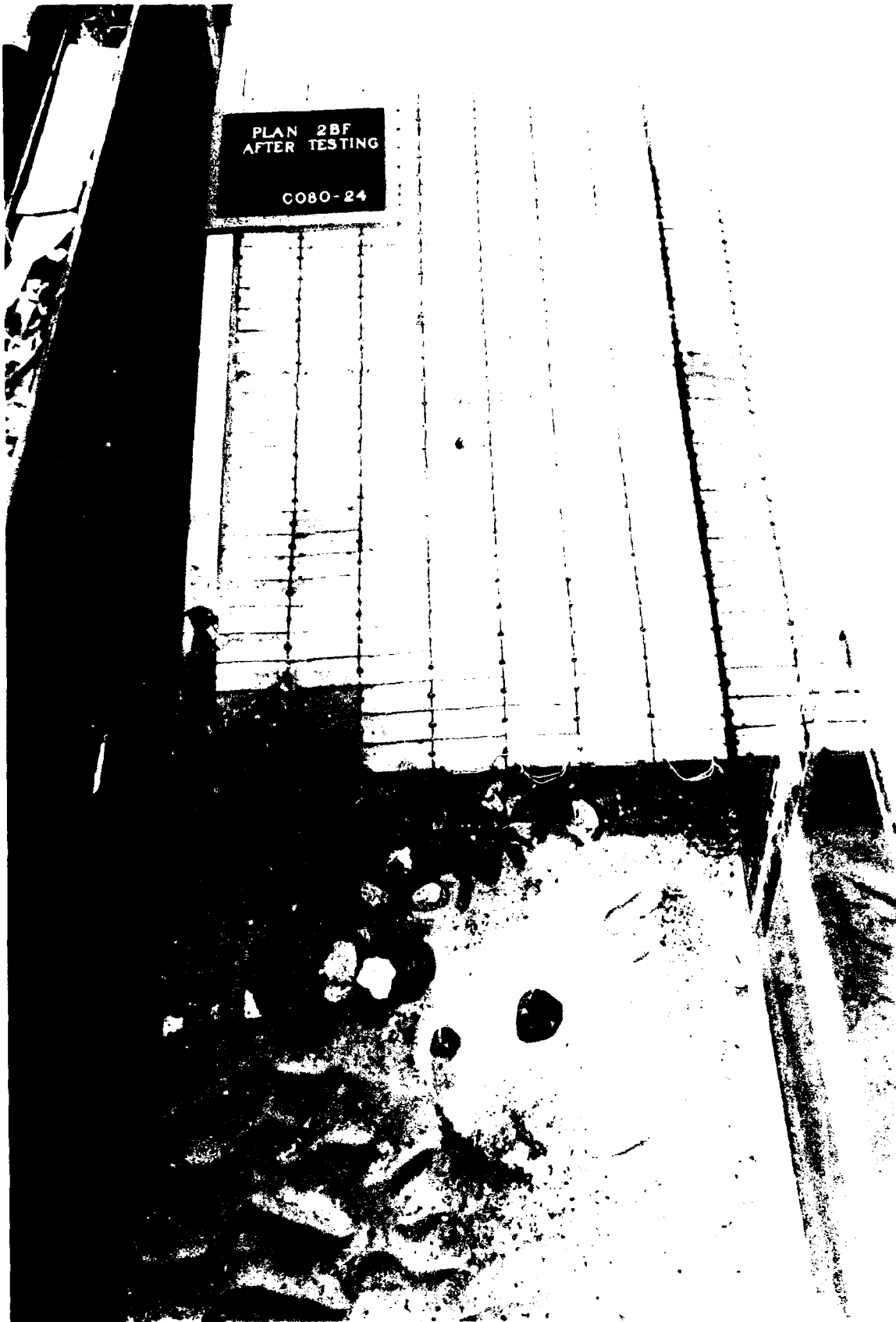
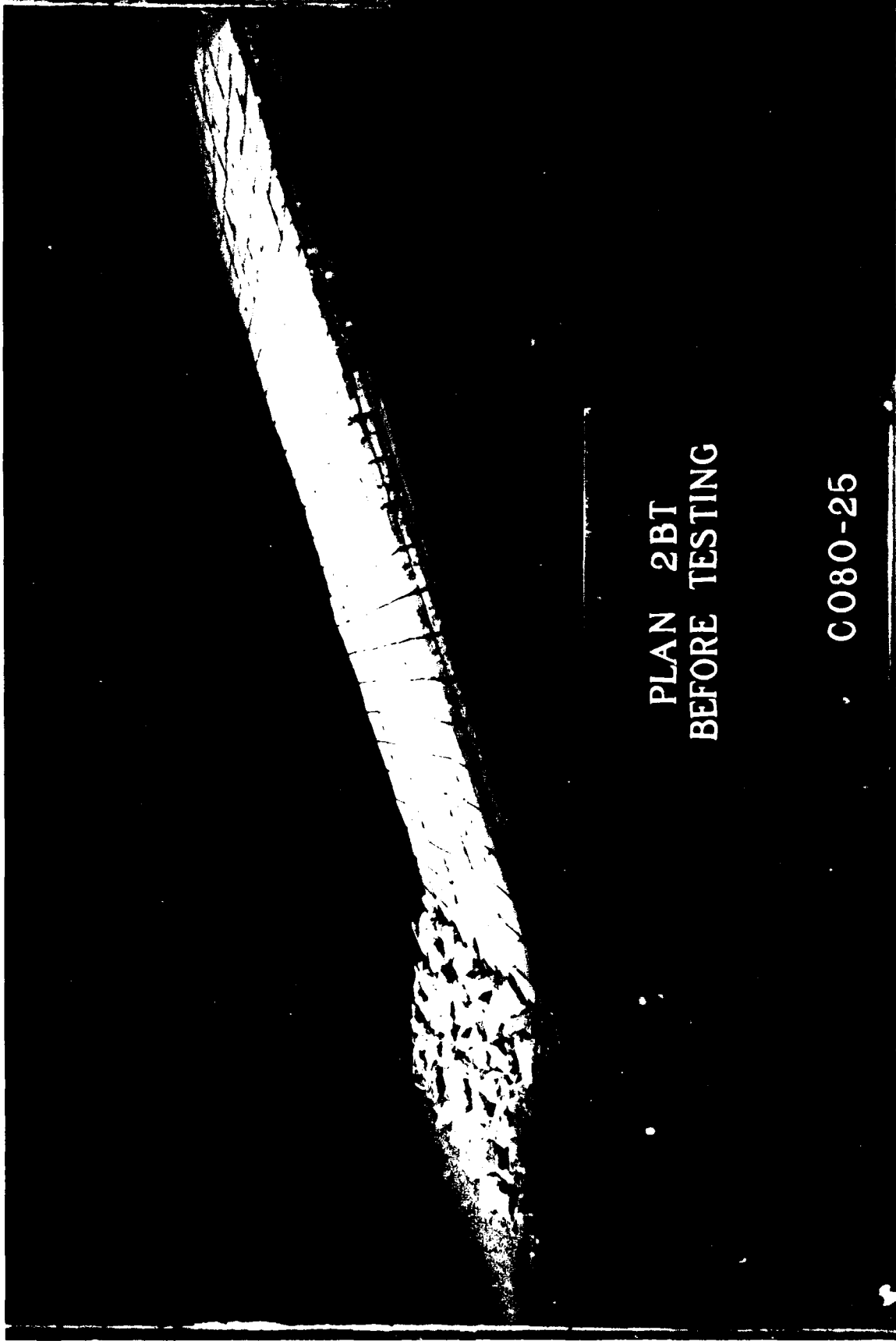


Photo 18. Sea-side view of Plan 2BF after wave attack



PLAN 2BT
BEFORE TESTING

C080-25

Photo 19. Side view of Plan 2BT before wave attack

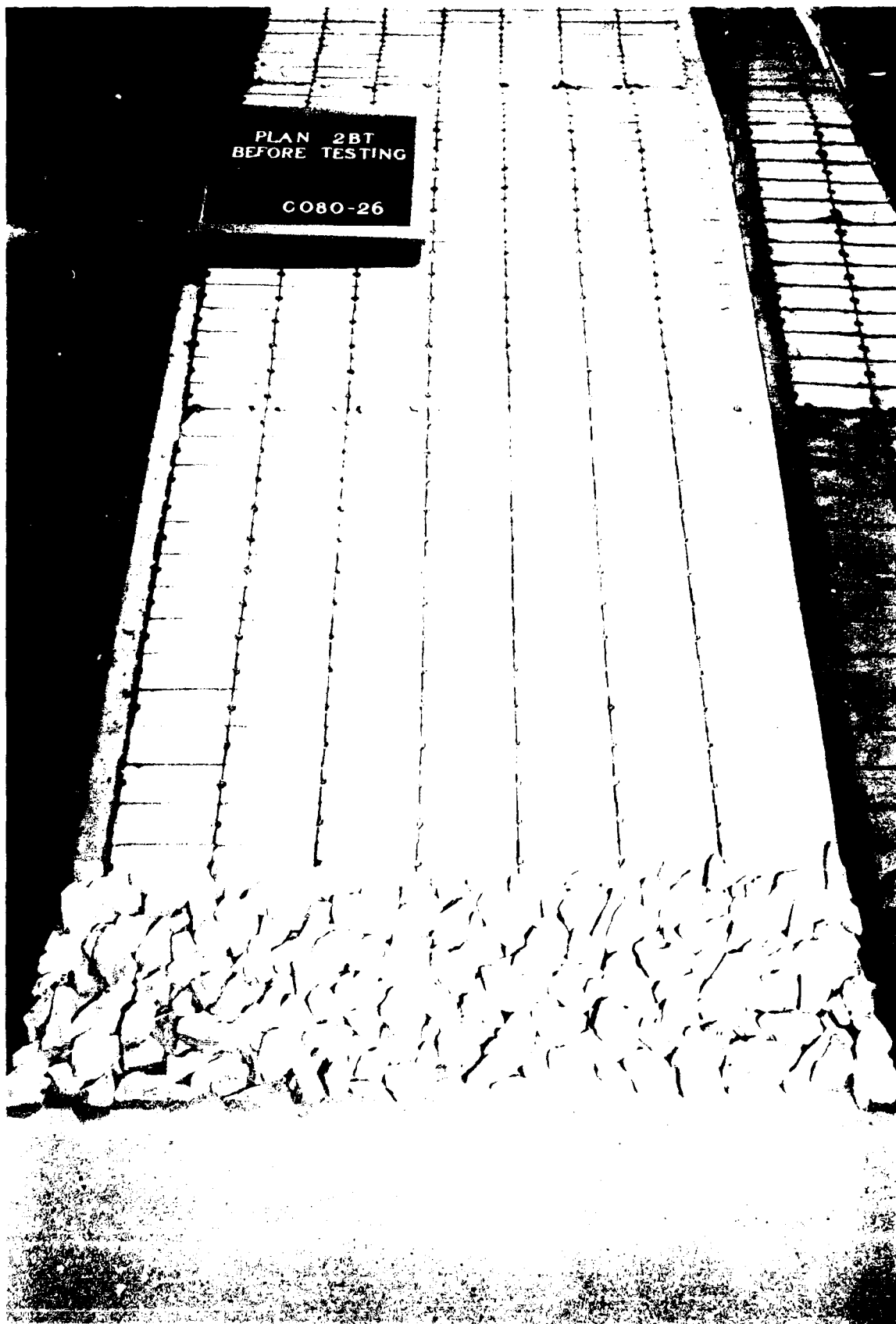
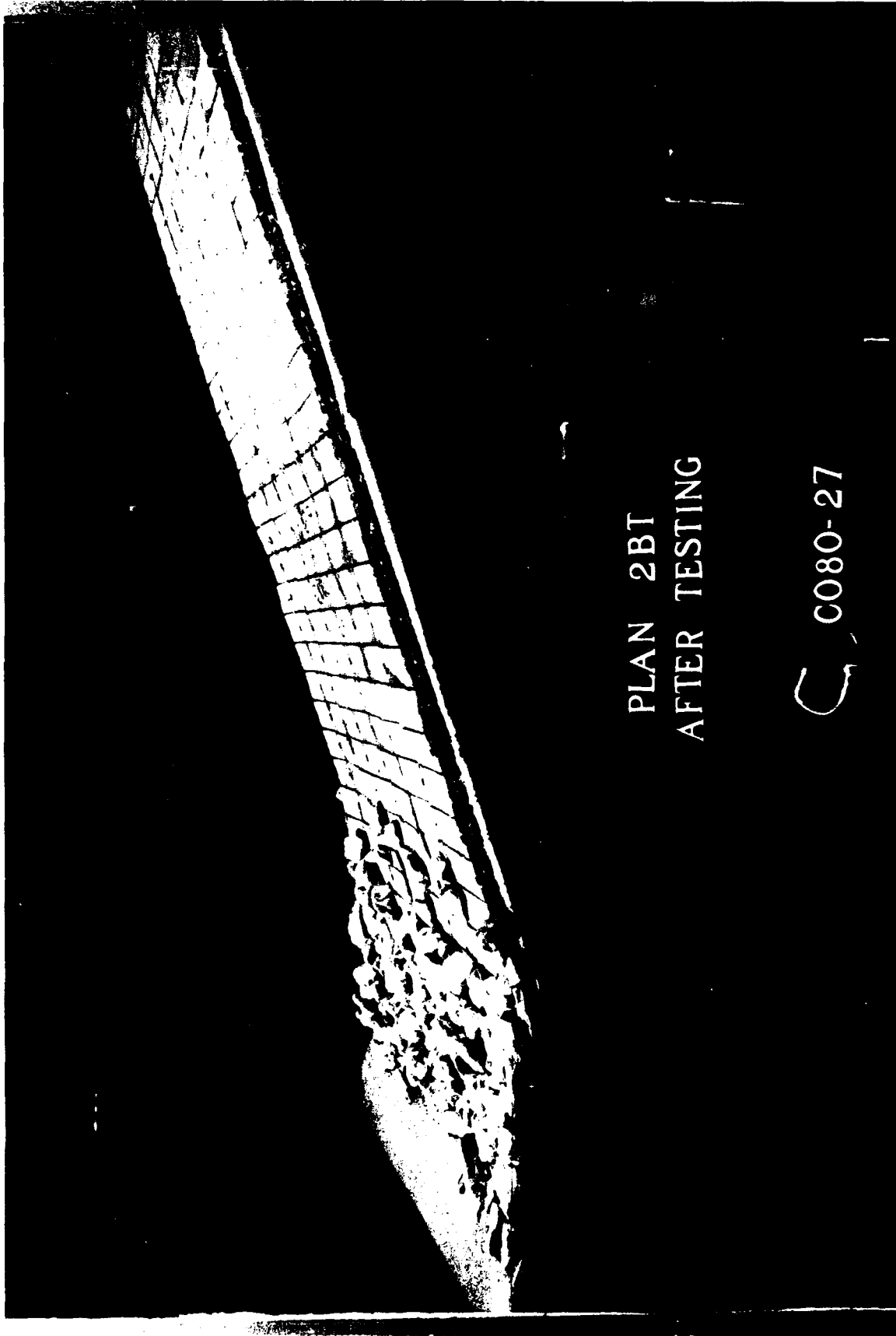


Photo 20. Sea-side view of Plan 2BT before wave attack



PLAN 2BT
AFTER TESTING

C, C080-27

Photo 21. Side view of Plan 2BT after wave attack



Photo 22. Sea-side view of Plan 2BT after wave attack

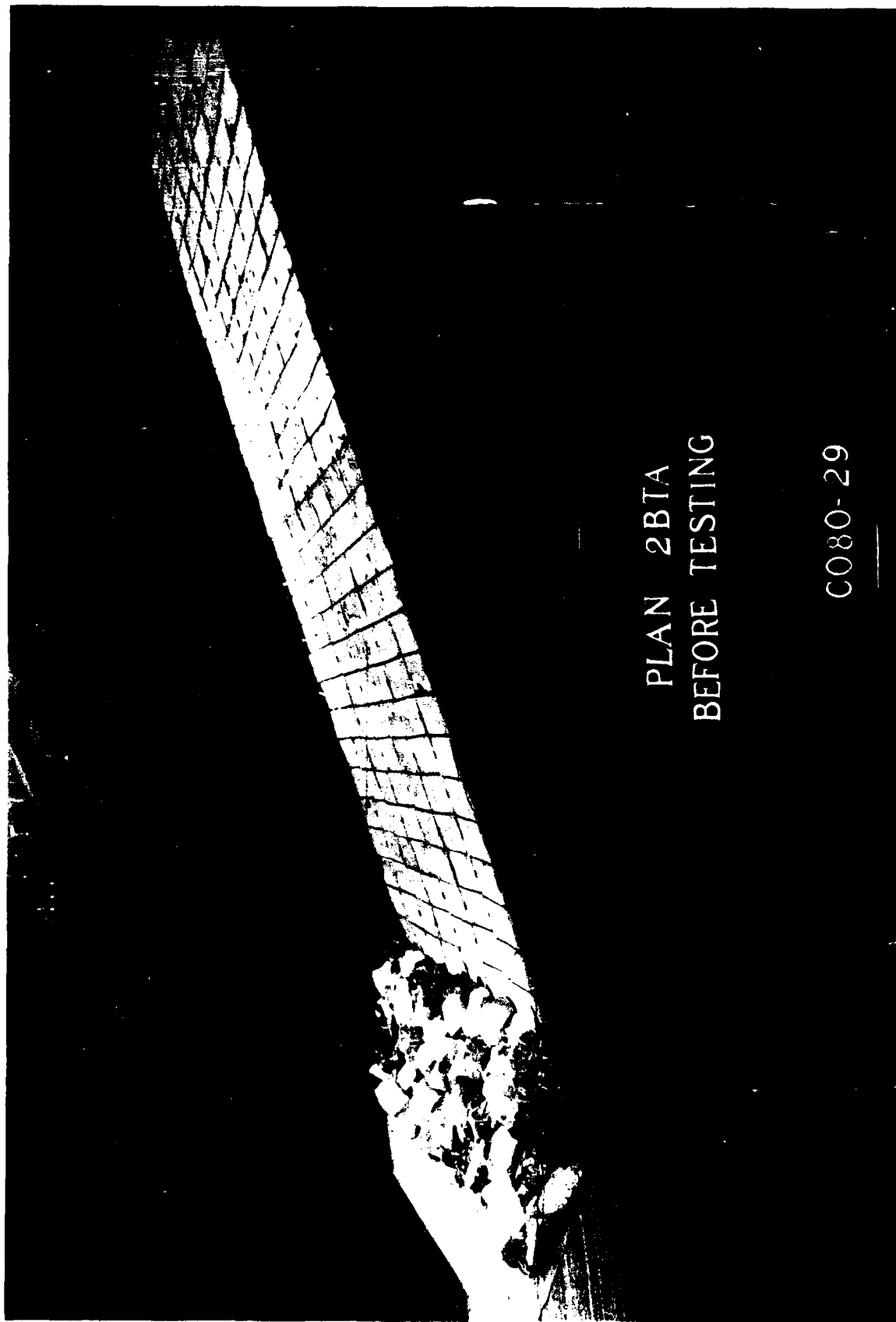


Photo 23. Side view of Plan 2BTA before wave attack



Photo 24. Sea-side view of Plan 2BTA before wave attack

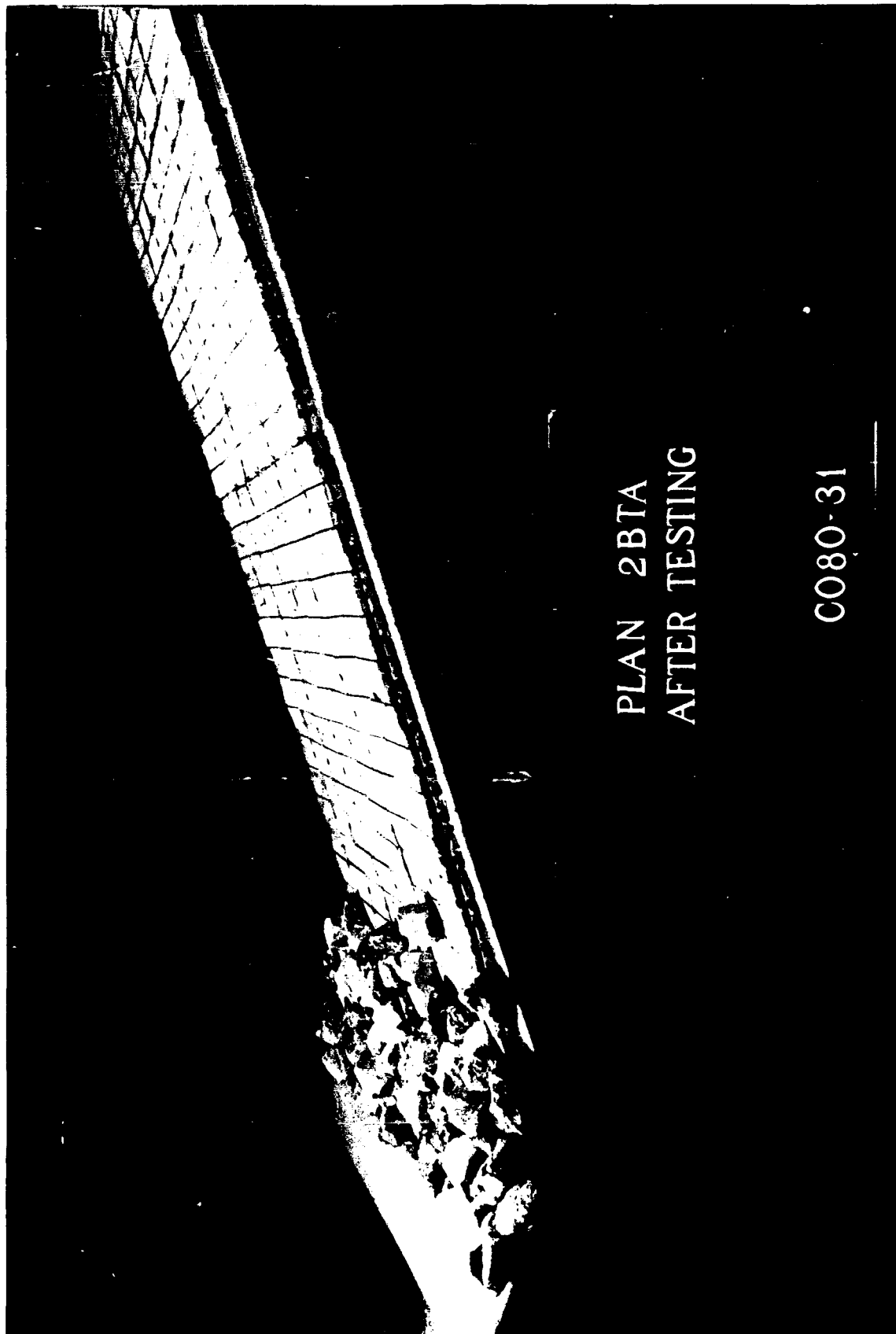
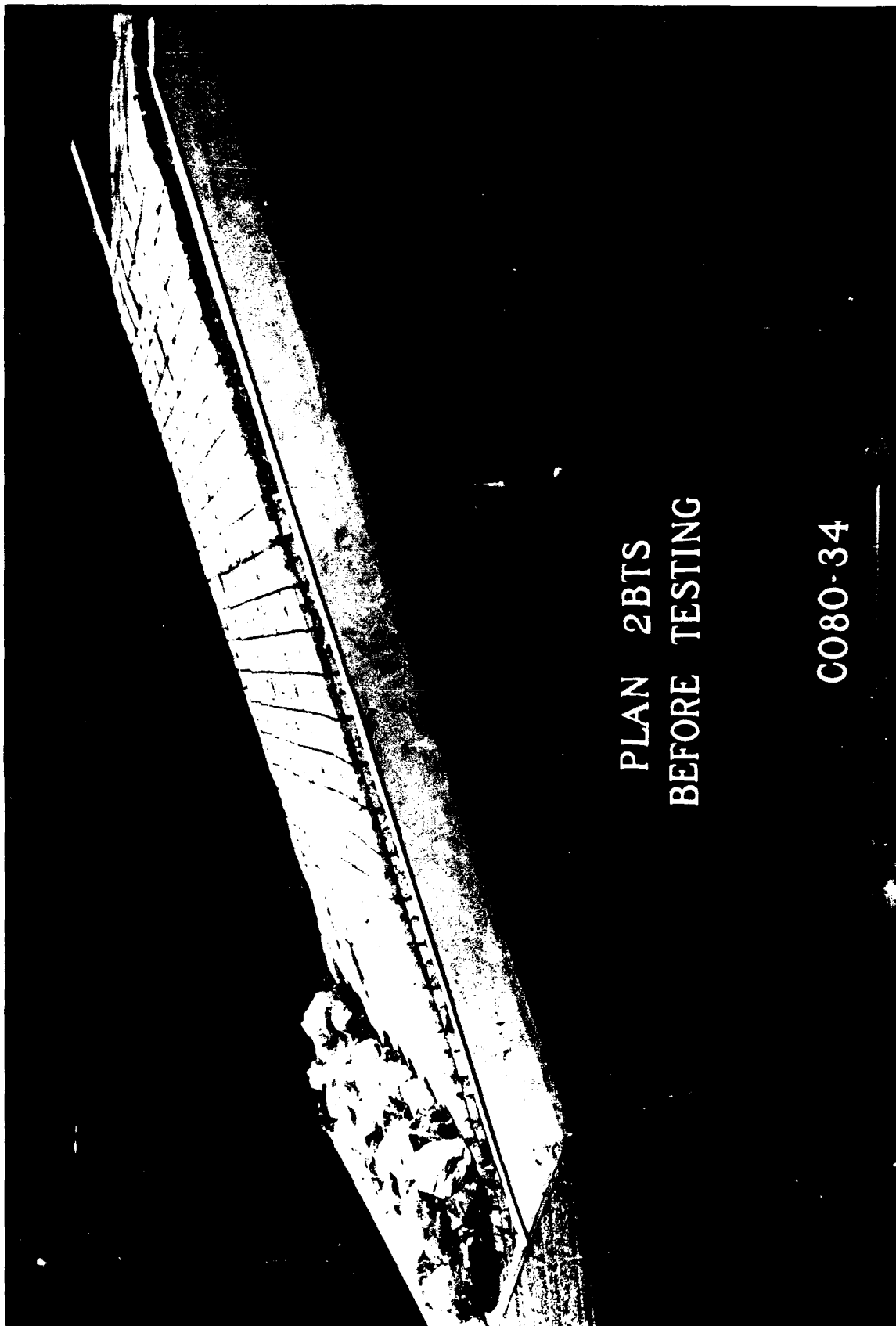


Photo 25. Side view of Plan 2BTA after wave attack



Photo 26. Sea-side view of Plan 2BTA after wave attack



PLAN 2BTS
BEFORE TESTING

C080-34

Photo 27. Side view of Plan 2BTS before wave attack

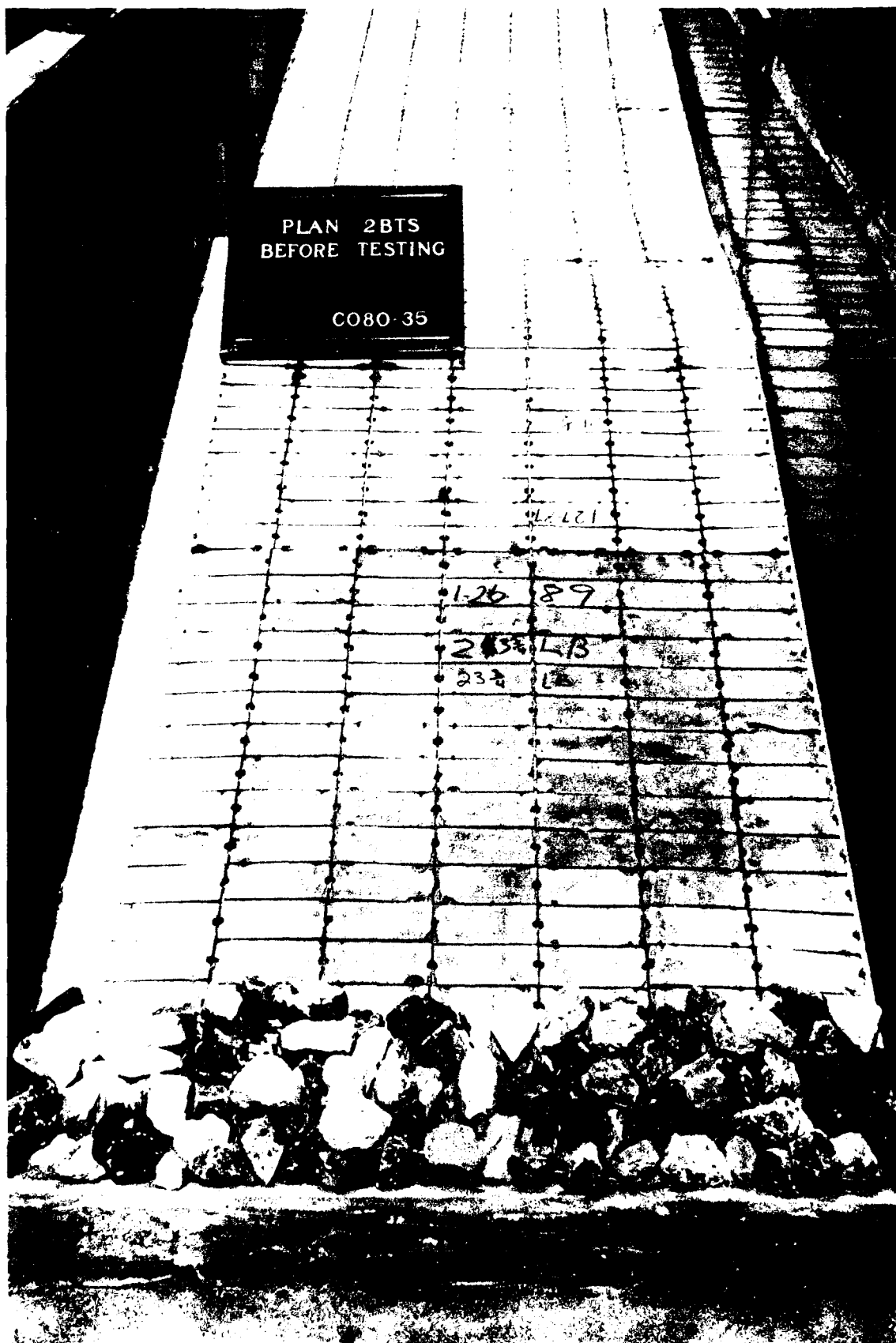
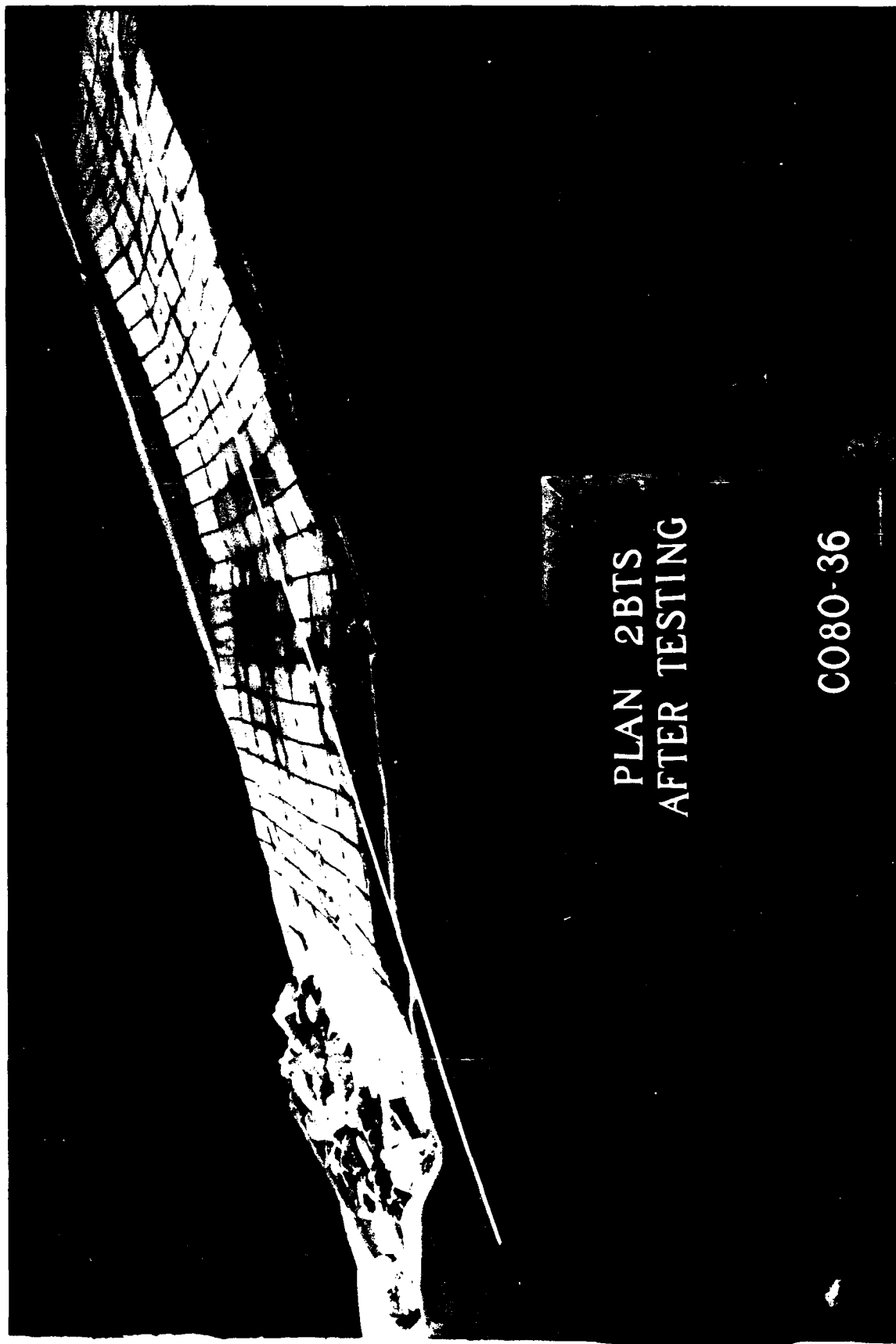


Photo 28. Sea-side view of Plan 2BTS before wave attack



PLAN 2BTS
AFTER TESTING

C080-36

Photo 29. Side view of Plan 2BTS after wave attack



Photo 30. Sea-side view of Plan 2BTS after wave attack

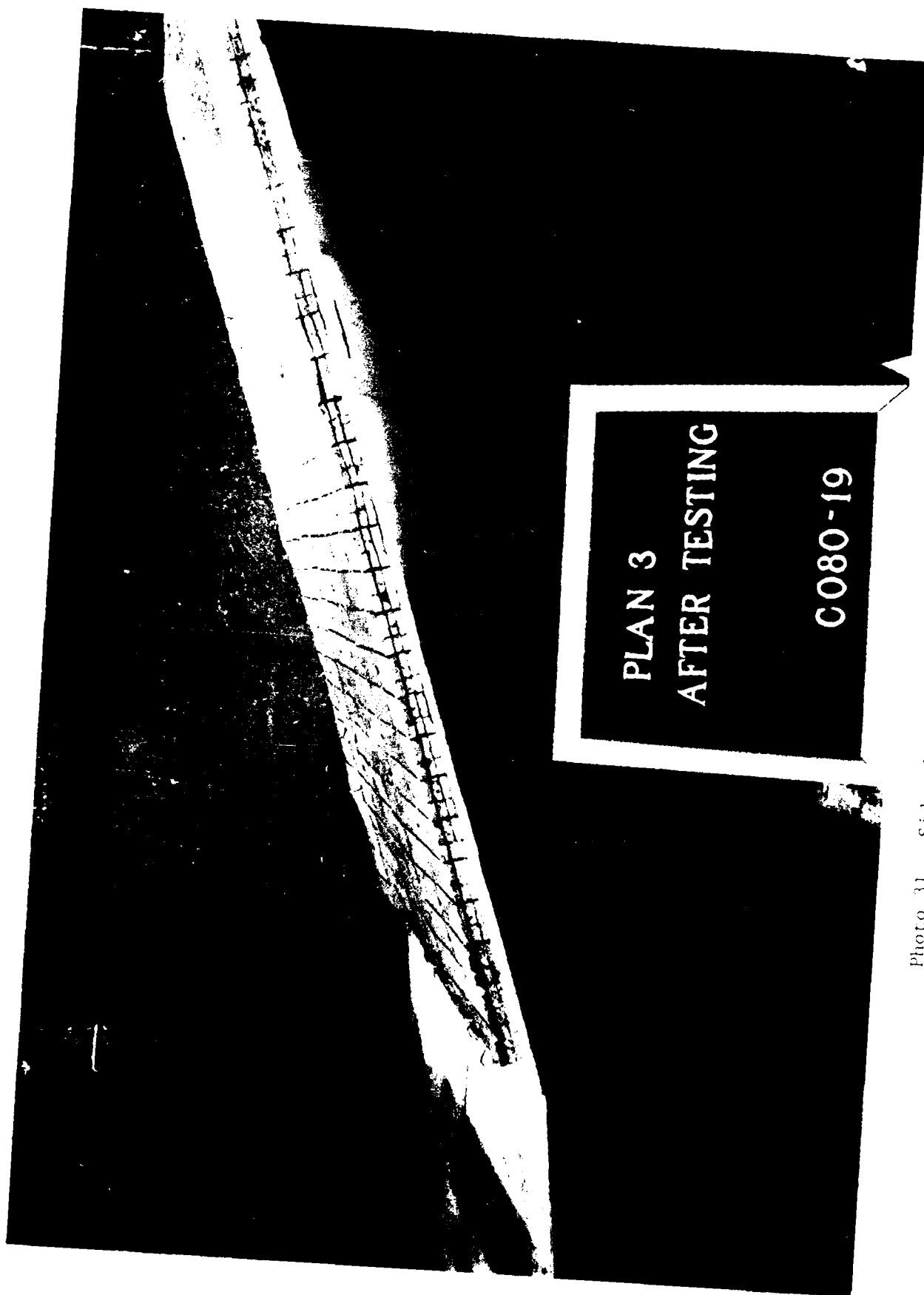


Photo 31. Side view of Plan 3 after wave attack

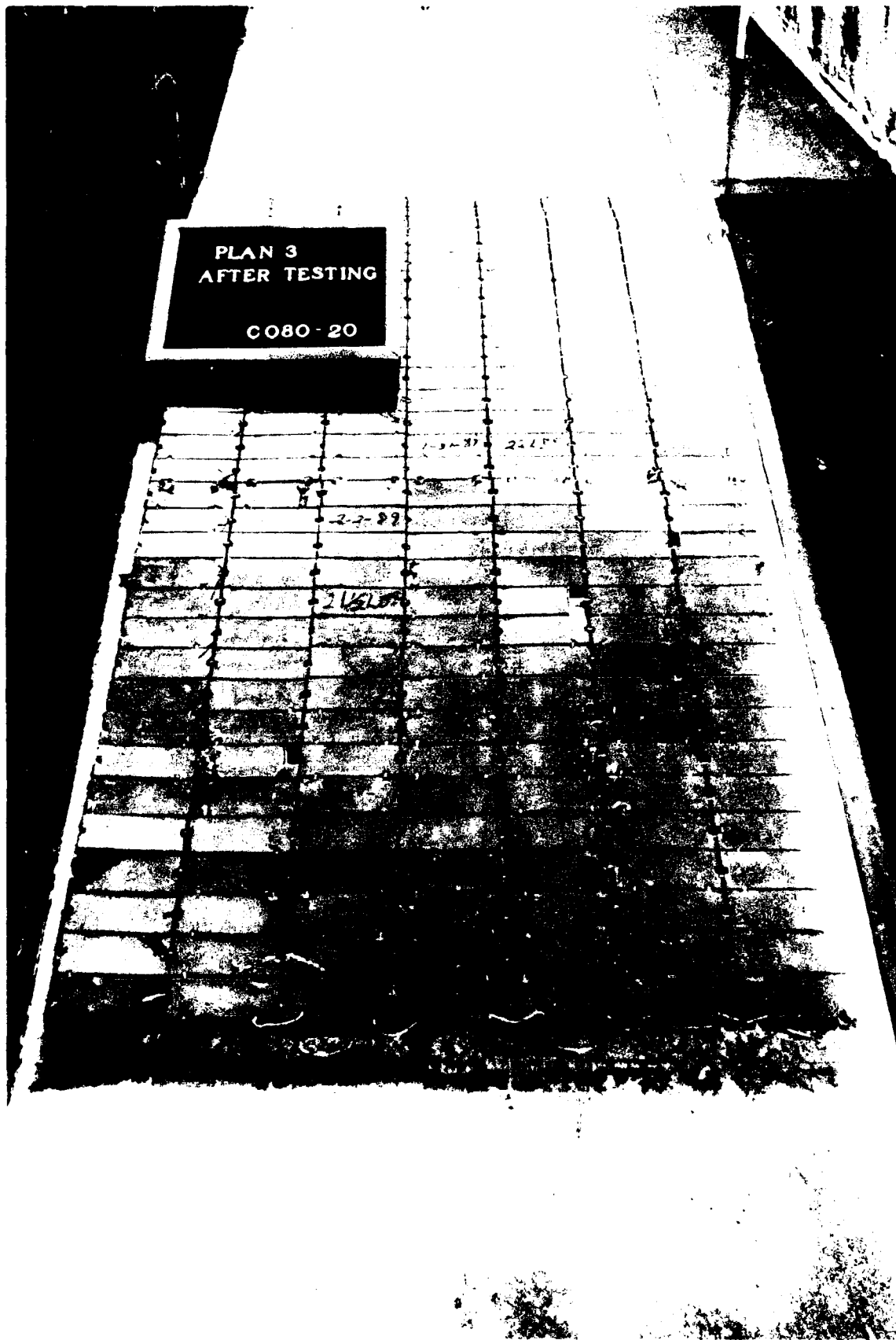


Photo 32. Sea-side view of Plan 3 after wave attack